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IMPACT IN RAILROAD BRIDGES

by E. J. Ruble, M. ASCE

STRUCTURAL DIVISION

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IMPACT IN RAILROAD BRIDGES

E. J. Ruble,[1] M. ASCE

SYNOPSIS

The subject of impact in railroad bridges has been of interest to designing engineers for over 100 years and many tests have been conducted to determine impact factors; however, it has only been since the development of the electrical strain gages within recent years that the stresses in structures under high speed trains could be reliably measured.

The paper describes the extensive field and laboratory investigations conducted by the Research Staff of the Association of American Railroads for Steel Concrete and Timber Railroad Bridges in the United States and presents a partial summary of its findings to date.

INTRODUCTION

Before presenting this paper, it would appear desirable to acquaint you with the organization known as the Association of American Railroads so that you can fully understand how the railroads conduct their joint research. The AAR is an organization of 198 railroads in the United States, Canada and Mexico, with a total of about 277,000 miles of track in operation. The activities of the AAR are controlled by an elected Board of Direction. The various departments of the Association, such as our operations and maintenance department, work on an annual budget, and this money is derived from the member railroads—the amount being based on their gross earnings. The general offices of the Association are located in Washington, D. C.; however, the Engineering Division office is in Chicago, and a new research center laboratory is now the headquarters for the research staff of this division.

The railroads as a whole form a vast laboratory for research under actual operating conditions and it is the policy of our staff, under the general direction of G. M. Magee, Research Engineer, to use this laboratory for the final evaluation of all our work. The research staff, of course, has a program investigating all phases of railroad engineering but only the program dealing with the 192,000 dynamically loaded steel, timber and reinforced concrete structures in the United States will be considered here.

The principal assignment of the research staff working on structural problems has been the determination of the behavior of structures under dynamic loads such as those produced by steam and diesel locomotives operating over a full range of speeds. The general purpose of this program is to secure factual data so that new bridges can be more economically designed and carrying capacity of the weaker bridges more safely determined.

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History

The first known discussion on the dynamic effects of moving loads over structures is to be found in the 1849 "Report of the Commissioner appointed to inquire into the Application of Iron to Railway Structures" and this report is based on the testimony of the most important civil engineers of the day. This English Parliamentary Commission conducted experiments and sponsored a theoretical study of the subject. While the Commission did not publish specifications, its report did have a pronounced influence on English and American specification writers for several years. Considering this fact, we are safe in saying that impact specifications were born in 1849.

In 1885, Professor S. W. Robinson, in his report to the Railroad and Telegraph Commissioners of Ohio, made a clear statement as to the various forces producing impact. Part of Professor Robinson's report dealt with the vibration of bridges. He not only recognized the effect of synchronous speed, but gave equations for computing the natural loaded frequencies of bridges.

The most complete series of tests made heretofore in this country were conducted by the Committee on Impact of the American Railway Engineering and Maintenance of Way Association under the direction of F. E. Turneaure, Dean of College of Engineering, University of Wisconsin. The results of these tests were published in 1911 and gave ample evidence of the effect of synchronous speed and the effect of locomotive counterbalancing. In addition to the test data, the report contains an interesting discussion on the theory of oscillations and span frequencies. Impact values, based on the data presented by Prof. Turneaure, were used in the design of railroad bridges until 1935.

In order to secure more data on the damping coefficients in bridges of this country, the late Mr. J. B. Hunley, Engineer of Bridges and Structures of the Big Four Railroad secured static and dynamic deflection readings on 39 different railroad spans under about 300 different locomotives. The report of these tests was published by the AREA in 1935 and a very thorough resume of the general theory of vibrations, as developed by Prof. S. Timoshenko, Prof. C. E. Inglis and others, is included. The report contains detailed results of the tests and also includes the dynamic magnifiers and damping coefficients obtained under the different classes of locomotives. The AREA design specifications for railroad bridges were revised in accordance with the recommendation for impact effects and was used until 1948.

In revising the design specification for impact in 1935, the bridge engineers realized that the entire subject of impact should be again studied on account of the increasing speeds and weights of the locomotives. They also realized that all previous specifications were based either on deflections or on mechanical instruments subject to considerable error due to vibrations. For these reasons they recommended that a Special AREA Committee on Impact be organized and funds be appropriated for the purchase of electrical recording instruments.

Instruments

The instruments used by the research staff to measure the static and dynamic strains in the railroad structures are of the electrical type now in common use for such work. The gages are either of the electro-magnetic type or the wire resistance type. The electro-magnetic gage is attached to the member in which the strains are to be measured by two special screws which are seated in two drilled and tapped holes, the center of these holes

being exactly two inches apart while the wire resistance gage is cemented to the steel surface. The gage on the steel is placed in one arm of an electrical bridge circuit while a similar gage or balancing gage is placed in the other arms of the circuit. In the electro-magnetic gage, a strain in the steel produces a corresponding change in the air gap between the two parts of the gage which produces a change in the electrical inpedance of the circuit. A strain in the steel under the wire resistance gage produces the same strain in the wires of the gage resulting in a resistance change of this gage.

The strain gages themselves are quite simple and inexpensive; however, the necessary electrical bridge circuits, power supply units, control units, amplifiers and oscillographs to record the strain from the dynamic loading of the structures are quite complicated and expensive. The research staff now has sufficient equipment so that the dynamic strains at 72 individual locations can be measured simultaneously, but it is very seldom that more than 24 circuits are ever used for any one test.

A large number of the railroad bridges selected for testing are accessible by highway, private roads or pasture so the truck with a special built body shown in Fig. 1 is used to house the recording instruments. It is not always possible to get as close to the bridge as shown for this particular bridge; however, sufficient lead wires are available so that the recording instruments can be about 500 ft from the gages on the steel.

The equipment shown in Fig. 2 is planned and arranged in the truck in such a manner that after the gages are mounted on the test members, the lead wires to the gages can be plugged into the amplifiers and the tests started with a minimum loss of time.

Many of the bridges selected for testing are inaccessible to our test truck so it becomes necessary to provide other housing facilities, as shown in Fig. 3, for the protection of the recording instruments.

The test building and recording instruments are subjected to considerable vibration when located on such a bridge but ample sponge rubber cushioning prevents any possible damage to the equipment as well as instrumental errors in the recordings.

A typical oscillogram is shown on the left of Fig. 4 while the location of the 12 gages on the 78-ft girder are shown on the right. The oscillogram was recorded under a Northern type steam locomotive at 53.4 mph. The ordinates or vertical movement of the traces on this oscillogram represent strain while the abscissa represents time as shown by the 0.01 second marks at the top and bottom.

The short length of oscillogram on the extreme left was taken before the locomotive was on the span and the short length on the right was taken after the locomotive and cars were off the span. Any vertical deviations from the straight lines joining these traces represent strain in the steel. A wheel trip on the rail at the center of the span connected electrically to a solenoid in the oscillograph placed a mark on the oscillogram every time a wheel passed over the tip. In this manner, the locomotive speed can be accurately determined as well as the position of the locomotive at time of any particular stress.

The first six traces on the oscillogram were produced by the strains in the web plates at the ends and quarter point of the girders. Trace 7 and 8 were produced by the strains in the lower flanges at the center of the span, and these two traces are excellent examples of forced vibration, the disturbing force being the unbalanced weights in the locomotive drivers. The period of these oscillations was the same as that of the driver. The last four traces were produced by the strains in the lower flange lateral bracing.

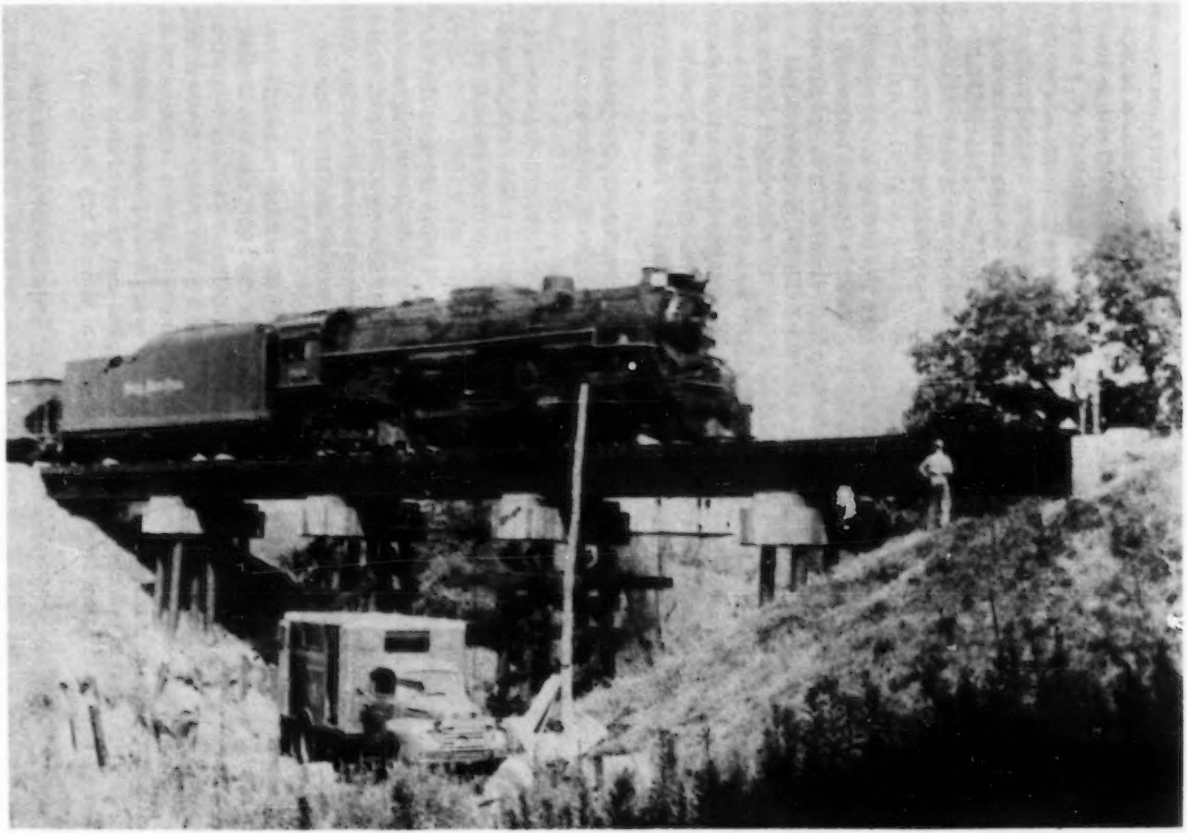


Fig. 1. View of Test Truck Housing Electrical Strain Gages.

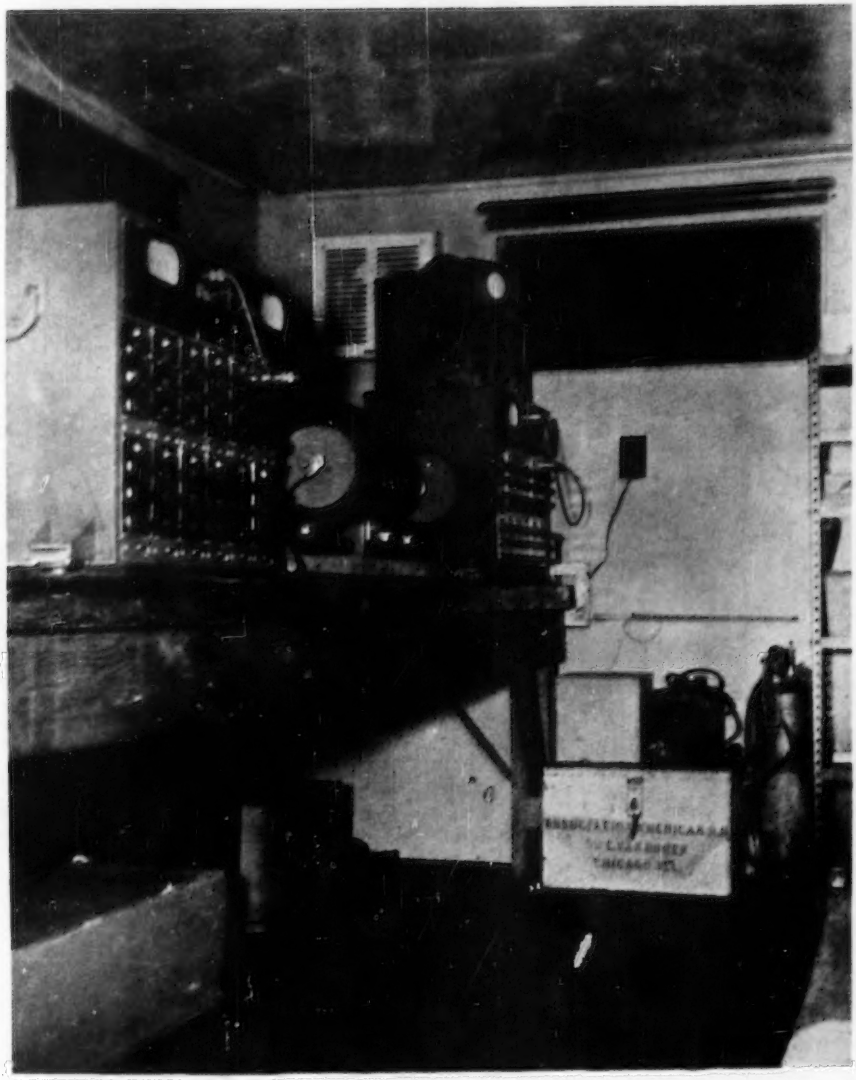


Fig. 2. View of Electrical Instruments Inside of Truck.

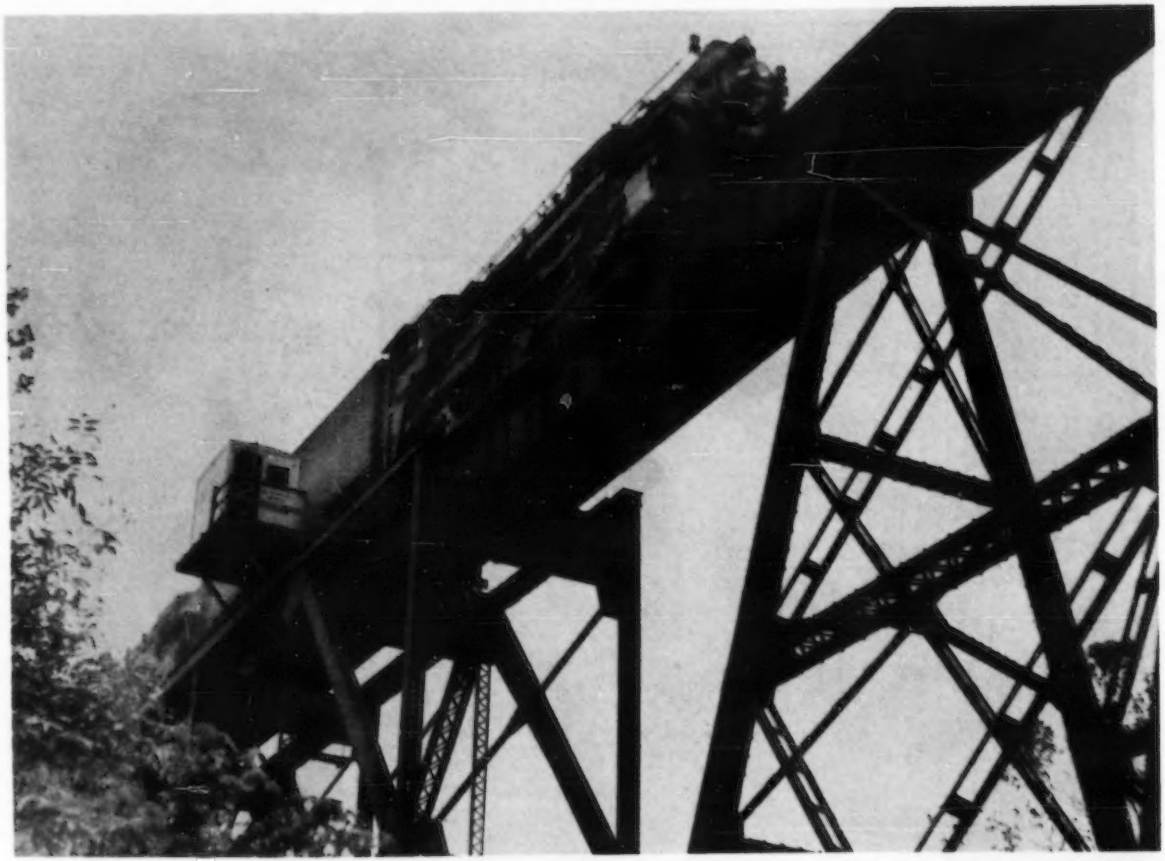


Fig. 3. View of Test House Housing Instruments.

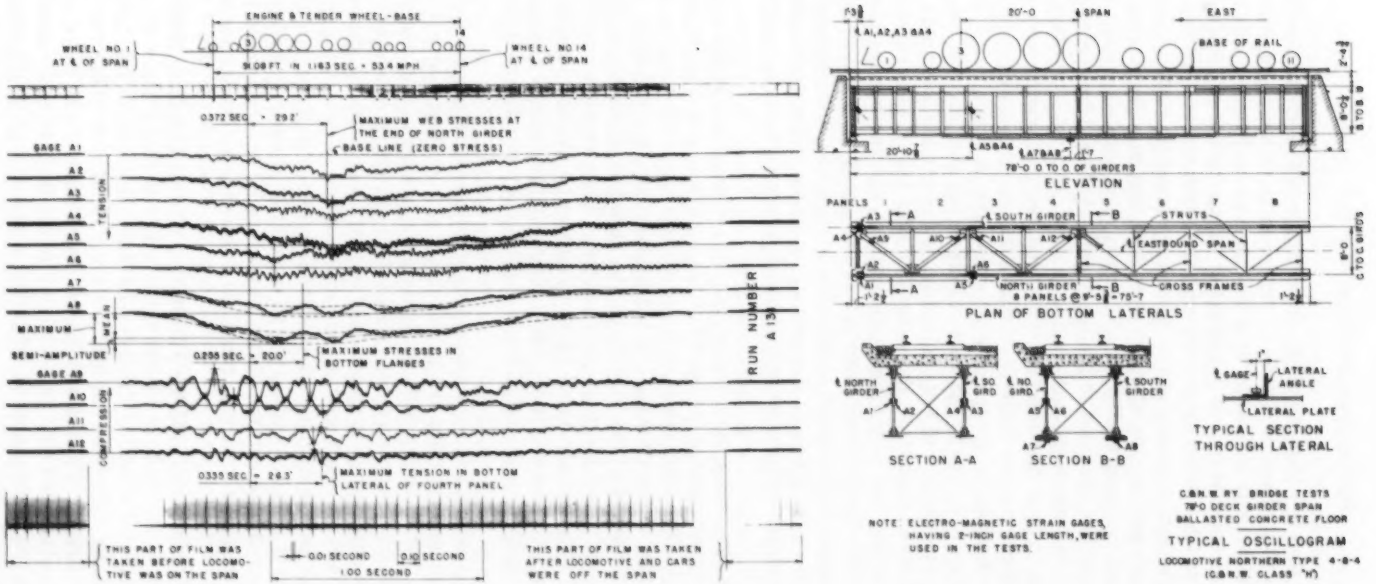


Fig. 4. Typical Oscillogram Showing Stresses Recorded in Railroad Girder Bridge.

Mechanical Oscillator Tests

At one time it was felt that the study of impact could be simplified by determining the damping characteristics of representative spans by measuring the strains and deflections produced by a mechanical oscillator. Accordingly, a large mechanical oscillator was constructed jointly by the engineering experiment station of Ohio State University and the research staff of the AAR. The oscillator consists basically of two identical eccentric rotors lying in the same vertical plane and turning at equal speeds in opposite directions and produces a vertical force of 19,200 lb. at 10 rps. The oscillator was so constructed that it could be fastened to the underside of the bridge, thus permitting the forced vibrations of the span under various loading conditions. The rotors are geared in such phase relation that the vertical components of the centrifugal forces exerted by their revolving unbalanced masses are in conjunction while the horizontal forces of each rotor are in opposition. The resultant external force is then in the vertical direction and ignoring the effect of small vertical movements of the oscillator, is the simple harmonic force.

The oscillator was mounted on the underside of a 102-ft 6-in truss span at the center of the span, and strain gages mounted on the various members of both trusses. In addition, deflectometers were mounted to measure the deflections of the center lower panel point. The bridge was then vibrated at various frequencies with different degrees of oscillator unbalance under dead load only, with coal cars on the bridge and with a heavy steam locomotive on the bridge.

The measured semi-amplitudes of deflection and stresses in the truss members were plotted against the speed of the rotors in revolutions per sec., as shown in Fig. 5. The semi-amplitudes of deflection plotted on this diagram were recorded under the dead weight of the bridge only, and it can be seen that the natural frequency of the bridge, or the speed at which the deflections became a maximum, was 9.20 cycles per second. This experimental value agreed very well with the calculated value as shown by the solid curve. The damping characteristics and dynamic magnifiers or the ratio of the measured semi-amplitudes to those calculated for a static load were obtained from these diagrams.

The results of these tests led to considerable discussion among the railroad bridge engineers regarding the practicability of the tests, and it was generally agreed that in future tests the dynamic magnifiers and damping characteristics should be determined from actual operating conditions whereby the deflections and strains are recorded under the passage of locomotives. The principal reason for making this decision was that in oscillator tests the disturbing force, which is usually the unbalanced forces in the driving wheels of the locomotive, is only applied a limited number of times. In addition to the limited number of applications of the disturbing force, the loaded frequency of vibration of the bridge is constantly changing as the locomotive crosses over.

It was further recognized that the vibrational effects resulting from the unbalanced forces in the driving wheels were only one of the sources of impact in bridges and that it would still be necessary to conduct field tests to determine the rolling effects of the locomotives, the effects resulting from uneven track, battered rail joints and out-of-round wheels and speed effects. However, it was recognized that an oscillator might be profitably used in the investigation of natural frequency of vibration of structures which are so complex as to make an estimate by computation difficult or unreliable. Such

an occasion might arise, for example, in the case of a tall building in which the installation of machinery was contemplated, or in the determination of critical frequencies of arches, draw bridges, continuous trusses or other complex structures.

Short Spans

There are approximately 36,000 short span steel railroad bridges in the United States today and up to 1941 the strains produced by moving locomotives has never been reliably measured in any of these bridges. Short span bridges of the type shown on Fig. 6 are subjected to considerable vibrational effect which produced instrumental errors in the mechanical type of strain gage used prior to the electrical type gage. All previous impact equations for spans of these lengths have therefore been based on assumptions and the allowances were made sufficiently large to be on the safe side.

In 1941, the research staff of the AAR started tests on spans of this type using electrical gages. The bridges selected for testing varied in length from 20 ft to 34 ft and readings were secured under 900 diesel and steam locomotives crossing over the bridges at speeds varying from 5 mph up to 85 mph for the steam locomotives and 100 mph for the diesels.

The maximum total impact percentages, based on the recorded static stresses, produced in the short spans by the diesel locomotives at various speeds are shown by the diagram on Fig. 7. The total impact values shown are the increase in strain over that produced by a slowly moving locomotive and results from all the effects previously mentioned. In general, there is a gradual increase in the magnitude of the total impact with an increase in speed up to about 40 mph. The impact appears to remain constant for speeds between 40 and 80 mph and then decreases with a further increase in speed. It can be seen that the impacts in the ballasted deck bridges, indicated by the solid circles, are about the same as those in the open deck bridges.

The maximum impacts produced in the same short spans by the steam locomotives at various speeds are shown on Fig. 8. Here again, there is a gradual increase in the magnitude of the total impact with an increase in speed up to about 40 mph with no further increase in speed up to 85 mph. In general, the maximum impacts produced by the diesel locomotives were about 48 percent while those produced by the steam locomotives were 76 percent with no appreciable difference between the impacts in open and ballasted deck bridges. This recorded impact value under the steam locomotives was considerably below the former design allowance of 108 per cent for the same length span.

In addition to the total impacts, the results of these tests on short spans were of particular interest to the railway bridge engineers for the following reasons:

1. The static stresses under both diesel and steam locomotives were lower than those usually calculated, indicating that as the span deflects, the axle loads are carried to the ends of the span by the frame action of the locomotive. The static stresses recorded under the diesel locomotives were lower than those recorded under modern steam locomotives since the weight of the diesel is spread over a greater length and the axle loads are somewhat lighter.

2. The maximum stresses in the bridges resulting from a wheel striking a battered rail joint at the center of the span were attained at a speed of 20 to

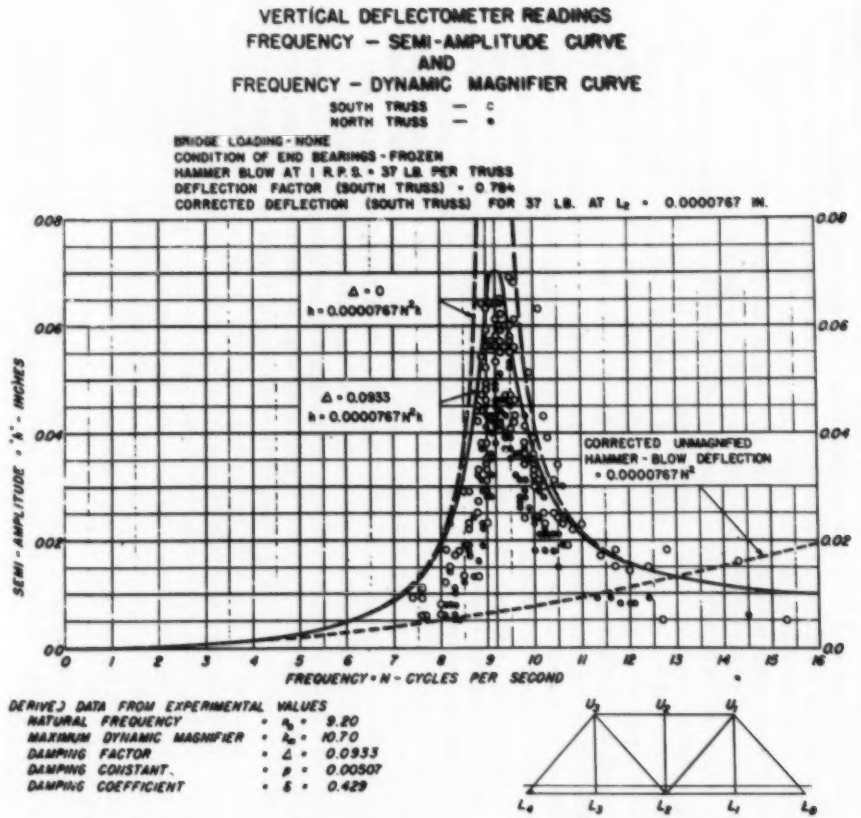

VERTICAL DEFLECTOMETER READINGS
FREQUENCY – SEMI-AMPLITUDE CURVE
AND
FREQUENCY – DYNAMIC MAGNIFIER CURVE
SOUTH TRUSS — c
NORTH TRUSS — •
BRIDGE LOADING - NONE
CONDITION OF END BEARINGS - FROZEN
HAMMER BLOW AT 1 R.P.S. = 37 LB. PER TRUSS
DEFLECTION FACTOR (SOUTH TRUSS) = 0.784
CORRECTED DEFLECTION (SOUTH TRUSS) FOR 37 LB. AT L2 = 0.0000767 IN.
Δ = 0
h = 0.0000767 N² h
Δ = 0.0933
h = 0.0000767 N² h
CORRECTED UNMAGNIFIED
HAMMER - BLOW DEFLECTION
= 0.0000767 N²
SEMI - AMPLITUDE = 'h' - INCHES
FREQUENCY = N - CYCLES PER SECOND
0.08
0.06
0.04
0.02
0.0
0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 N
DERIVED DATA FROM EXPERIMENTAL VALUES
NATURAL FREQUENCY = n_0 = 9.20
MAXIMUM DYNAMIC MAGNIFIER = h_m = 10.70
DAMPING FACTOR = Δ = 0.0933
DAMPING CONSTANT = β = 0.00507
DAMPING COEFFICIENT = δ = 0.429
U3 U2 U1
L4 L3 L2 L1 L0

Fig. 5. Diagram of Deflections Produced by a Mechanical Oscillator.

Fig. 6. General View of Short Beam Span on Steel Pile Bents.

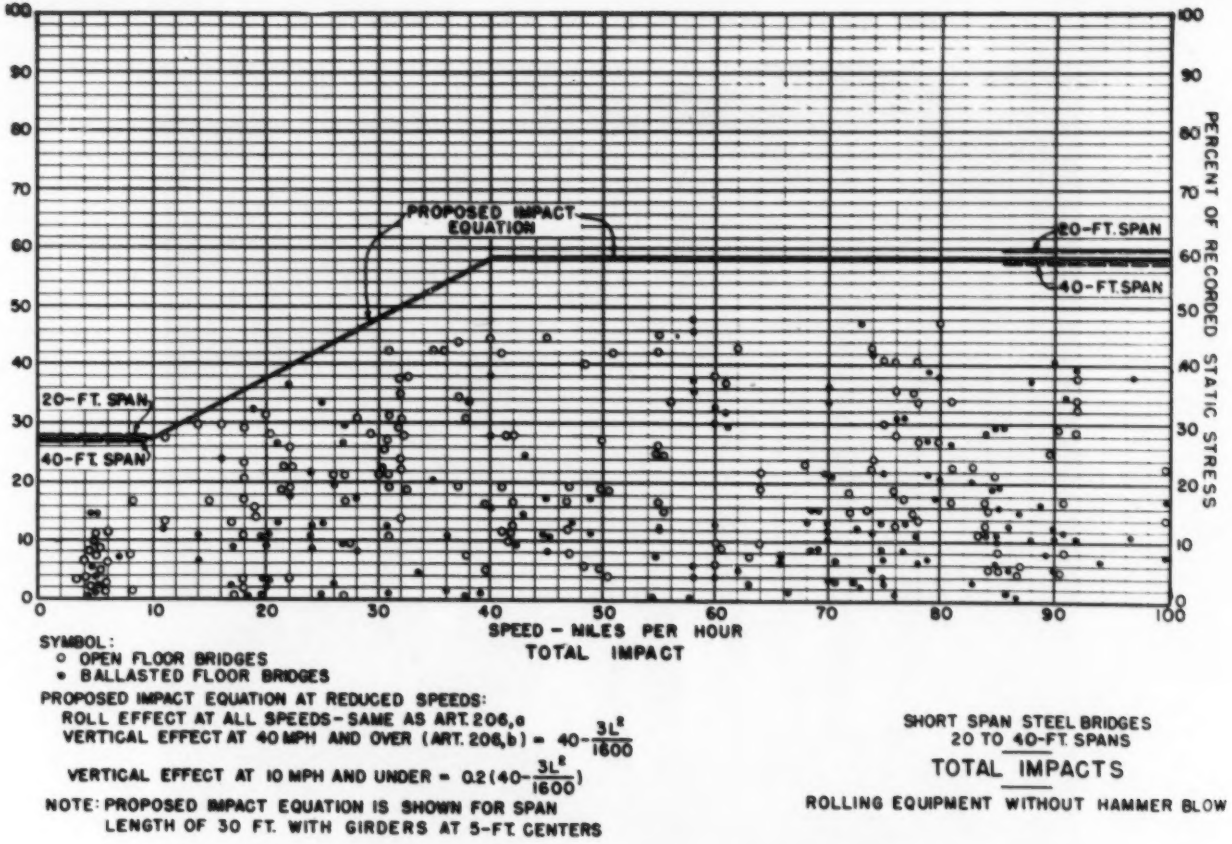

PROPOSED IMPACT EQUATION
20-FT. SPAN
40-FT. SPAN
20-FT. SPAN
40-FT. SPAN
PERCENT OF RECORDED STATIC STRESS
SPEED - MILES PER HOUR
TOTAL IMPACT
SYMBOL:
○ OPEN FLOOR BRIDGES
• BALLASTED FLOOR BRIDGES
PROPOSED IMPACT EQUATION AT REDUCED SPEEDS:
ROLL EFFECT AT ALL SPEEDS - SAME AS ART. 206,a
VERTICAL EFFECT AT 40 MPH AND OVER (ART. 206,b) = $40-\frac{3L^2}{1600}$
VERTICAL EFFECT AT 10 MPH AND UNDER = $0.2(40-\frac{3L^2}{1600})$
NOTE: PROPOSED IMPACT EQUATION IS SHOWN FOR SPAN LENGTH OF 30 FT. WITH GIRDERS AT 5-FT. CENTERS
SHORT SPAN STEEL BRIDGES
20 TO 40-FT. SPANS
TOTAL IMPACTS
ROLLING EQUIPMENT WITHOUT HAMMER BLOW

Fig. 7. Recorded Impacts in Short Span Bridges Produced by Diesel Locomotives.

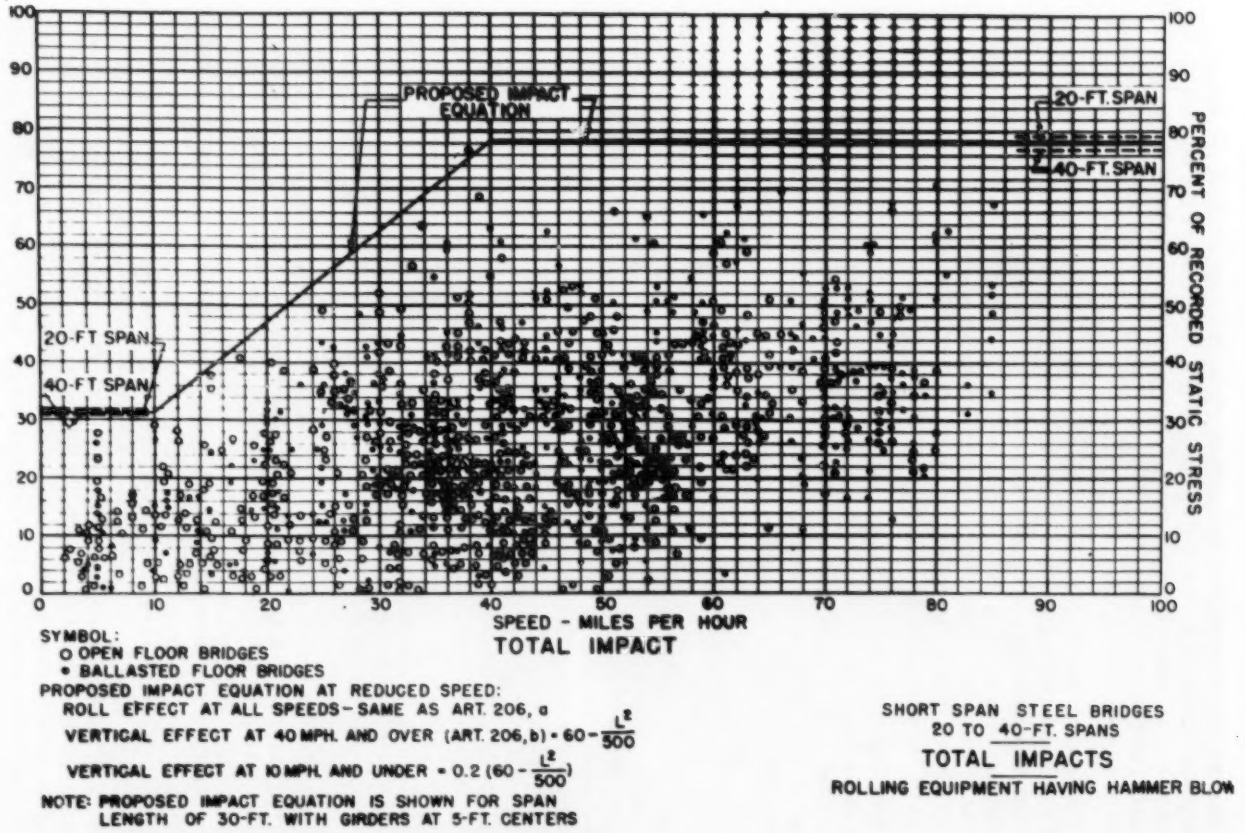

PROPOSED IMPACT EQUATION
20-FT. SPAN
40-FT. SPAN
20-FT SPAN
40-FT SPAN
PERCENT OF RECORDED STATIC STRESS
SPEED - MILES PER HOUR
TOTAL IMPACT
SYMBOL:
O OPEN FLOOR BRIDGES
• BALLASTED FLOOR BRIDGES
PROPOSED IMPACT EQUATION AT REDUCED SPEED:
ROLL EFFECT AT ALL SPEEDS - SAME AS ART. 206, a
VERTICAL EFFECT AT 40 MPH. AND OVER (ART. 206, b) = 60 - L²/500
VERTICAL EFFECT AT 10 MPH. AND UNDER = 0.2 (60 - L²/500)
NOTE: PROPOSED IMPACT EQUATION IS SHOWN FOR SPAN LENGTH OF 30-FT. WITH GIRDERS AT 5-FT. CENTERS
SHORT SPAN STEEL BRIDGES
20 TO 40-FT. SPANS
TOTAL IMPACTS
ROLLING EQUIPMENT HAVING HAMMER BLOW

Fig. 8. Recorded Impacts in Short Span Bridges Produced by Steam Locomotives.

30 mph and became smaller as the speed of the locomotive increased. At speeds of 100 mph the effect of the battered rail joint was negligible.

3. The use of 1/2-in rubber fabric pads under the regular steel tie plates and placed on top of the bridge or track ties did not reduce the amount of impact in these short span bridges. While no reduction in impact can be realized by the use of pads, they do reduce the mechanical wear on the tie, thus increasing the tie life.

4. There was considerable variation in the recorded stresses produced by the various types of locomotives and even by the same type of locomotive so that only about 10 per cent of the trains passing over the short spans produced stresses varying from 90 per cent of the maximum, to the maximum. About 50 per cent of the trains produced stresses whose magnitude were about 60 per cent of the maximum recorded. This variation in the stresses must be considered in determining the fatigue life of the structure.

5. The elasticity of the supports plays an important part in the magnitude of the stresses in the structure. In the particular bridge shown on Fig. 6 the stresses in the center spans supported on the long piles were considerably smaller than those in the end span supported on the more rigid concrete abutment.

Girder Spans

With the completion of the investigation of the stresses and impacts in short span bridges, the research staff began its investigation of girder spans such as shown on Fig. 9 and data has now been assembled on thirty-seven girder spans, varying in length from 40 ft. to 140 ft, and readings have been secured under about 3200 diesel and steam locomotives.

Considerable progress has been made in analyzing the data secured on these various length girder spans. A summary of the total impacts produced by the diesel locomotives on several spans from 40 ft to 60 ft in length is shown on Fig. 10. The total impacts recorded in the open floor bridges are shown by the open-circle symbol while those recorded in the bridges with the ballasted floor are shown by the solid circle. It appears that the impacts in the ballasted floor bridges are considerably lower than those recorded in the open floor bridges but this could be due to the larger number of readings taken on the open deck bridges.

The total impacts recorded by the steam locomotives in the same girder spans are shown on Fig. 11 and it is apparent that there is very little difference in the impacts recorded in the open and ballasted floor bridges. The proposed impact equations, which have now been adopted by the railroad bridge engineers, represent a considerable reduction over that formerly used for the shorter girder spans but is about the same for the longer girder spans.

The maximum total impact percentages, based on the recorded static stresses, produced by the diesel locomotives in girder spans varying in length from 90 to 140 ft, are shown in Fig. 13. It appears that the impacts increase gradually up to a speed of 60 mph but the speed of 40 mph was selected for the proposed equation to agree with other span lengths.

The maximum total impacts recorded during the passage of the steam locomotives over the same 90 to 140 ft spans are shown on Fig. 14 and it is quite evident that a speed of 30 to 40 mph was the critical speed for the steam locomotives on these longer spans. In the earlier studies of impact or dynamic effects in railroad bridges by our staff, it was generally expected that the

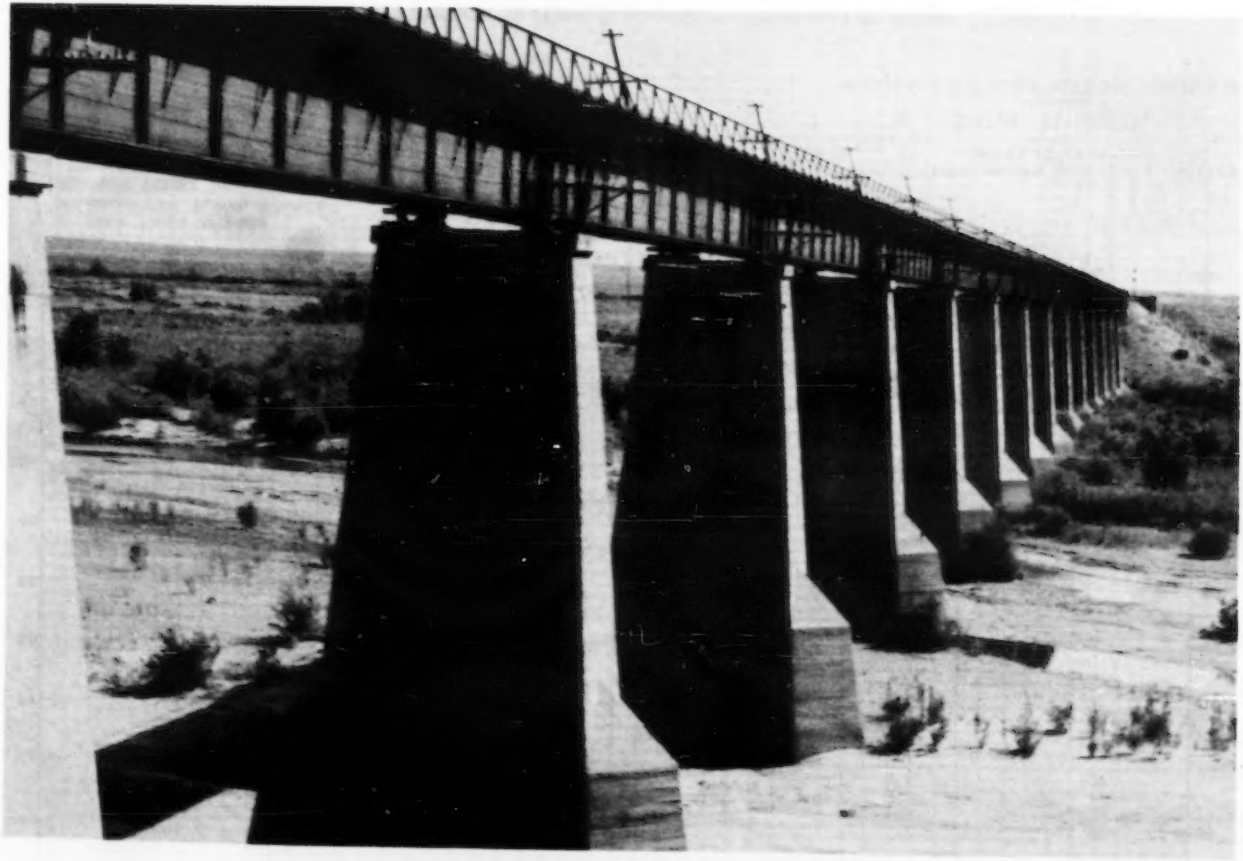


Fig. 9. General View of Girder Spans on Concrete Piers.

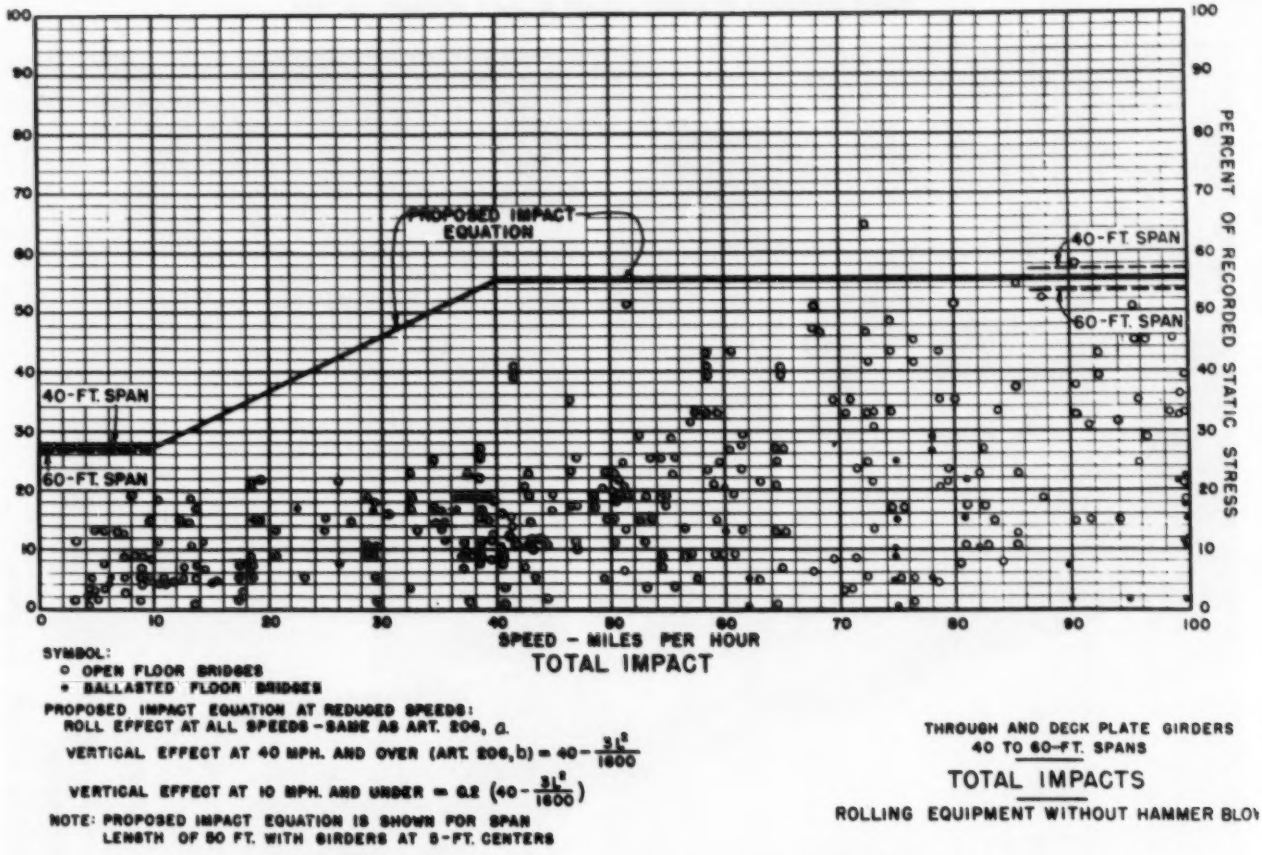

PROPOSED IMPACT EQUATION
40-FT. SPAN
60-FT. SPAN
40-FT. SPAN
60-FT. SPAN
PERCENT OF RECORDED STATIC STRESS
SPEED - MILES PER HOUR
TOTAL IMPACT
SYMBOL:
○ OPEN FLOOR BRIDGES
• BALLASTED FLOOR BRIDGES
PROPOSED IMPACT EQUATION AT REDUCED SPEEDS:
ROLL EFFECT AT ALL SPEEDS - SAME AS ART. 206, a.
VERTICAL EFFECT AT 40 MPH. AND OVER (ART. 206, b) = 40 - 31²/1600
VERTICAL EFFECT AT 10 MPH. AND UNDER = 0.2 (40 - 31²/1600)
NOTE: PROPOSED IMPACT EQUATION IS SHOWN FOR SPAN LENGTH OF 50 FT. WITH GIRDERS AT 8-FT. CENTERS
THROUGH AND DECK PLATE GIRDERS
40 TO 60-FT. SPANS
TOTAL IMPACTS
ROLLING EQUIPMENT WITHOUT HAMMER BLOW

Fig. 10. Recorded Impacts in Short Girder Sp... Produced by Diesel Locomotives.

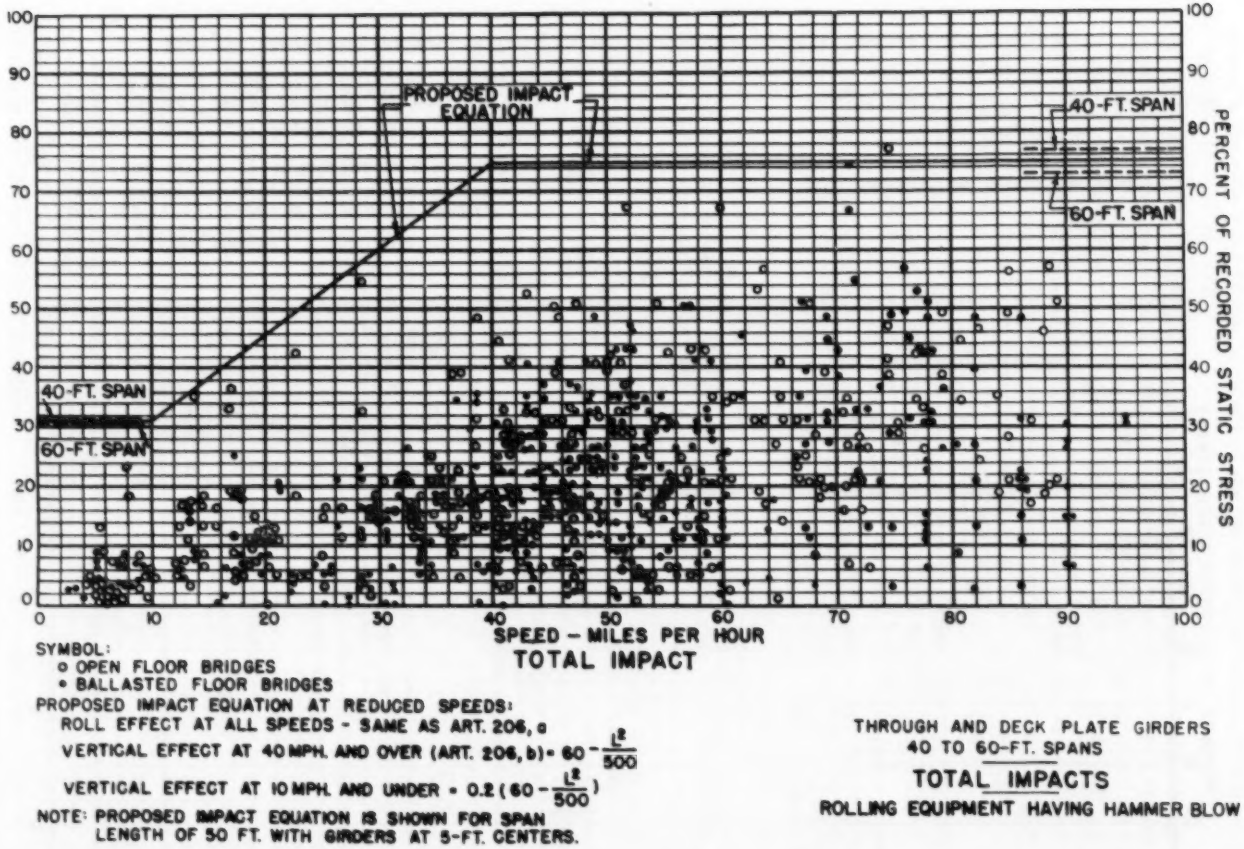

PROPOSED IMPACT EQUATION
40-FT. SPAN
60-FT. SPAN
40-FT. SPAN
60-FT. SPAN
PERCENT OF RECORDED STATIC STRESS
SPEED – MILES PER HOUR
TOTAL IMPACT
SYMBOL:
o OPEN FLOOR BRIDGES
• BALLASTED FLOOR BRIDGES
PROPOSED IMPACT EQUATION AT REDUCED SPEEDS:
ROLL EFFECT AT ALL SPEEDS - SAME AS ART. 206, a
VERTICAL EFFECT AT 40 MPH. AND OVER (ART. 206, b) = 60 - L²/500
VERTICAL EFFECT AT 10 MPH. AND UNDER = 0.2 (60 - L²/500)
NOTE: PROPOSED IMPACT EQUATION IS SHOWN FOR SPAN LENGTH OF 50 FT. WITH GIRDERS AT 5-FT. CENTERS.
THROUGH AND DECK PLATE GIRDERS
40 TO 60-FT. SPANS
TOTAL IMPACTS
ROLLING EQUIPMENT HAVING HAMMER BLOW

Fig. 11. Recorded Impacts in Short Girder Spans Produced by Steam Locomotives.

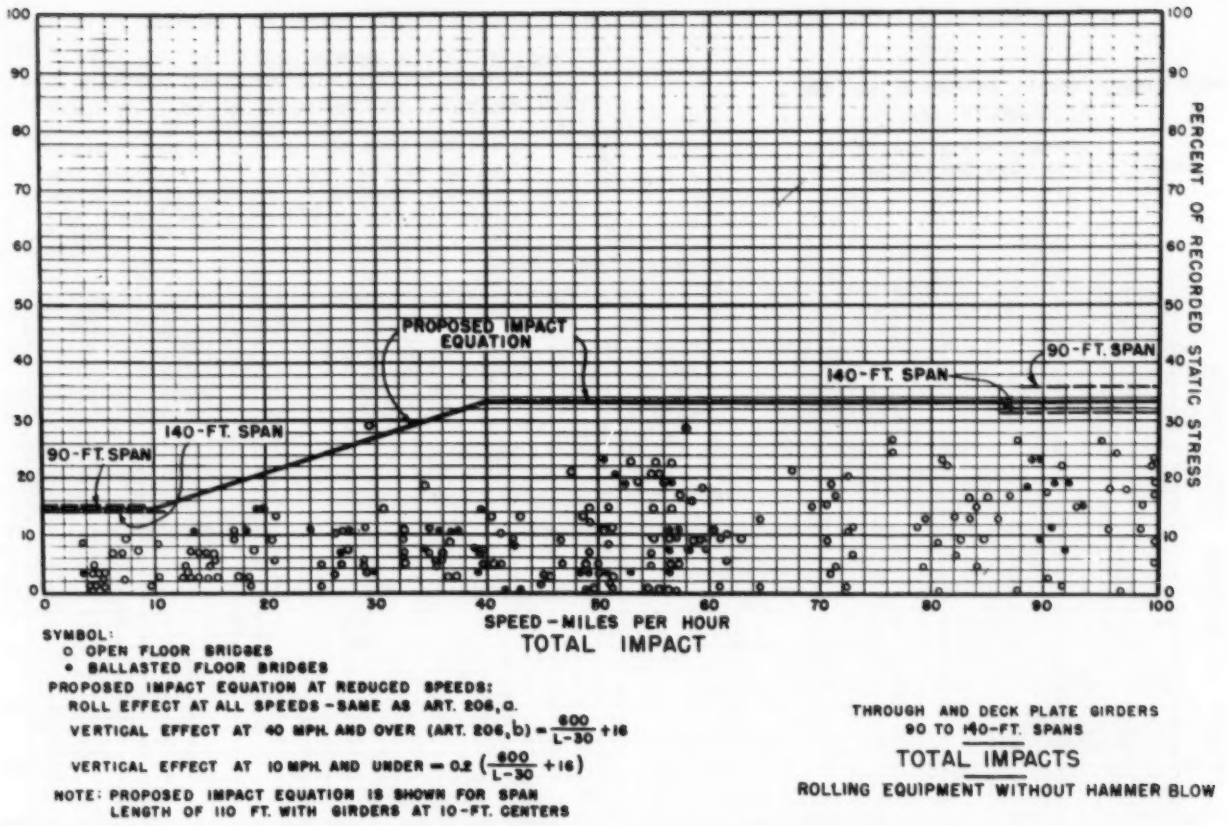

PROPOSED IMPACT EQUATION
140-FT. SPAN
90-FT. SPAN
140-FT. SPAN
90-FT. SPAN
PERCENT OF RECORDED STATIC STRESS
SPEED - MILES PER HOUR
TOTAL IMPACT
SYMBOL:
o OPEN FLOOR BRIDGES
• BALLASTED FLOOR BRIDGES
PROPOSED IMPACT EQUATION AT REDUCED SPEEDS:
ROLL EFFECT AT ALL SPEEDS - SAME AS ART. 206, a.
VERTICAL EFFECT AT 40 MPH AND OVER (ART. 206, b) = 600/(L-30) + 16
VERTICAL EFFECT AT 10 MPH AND UNDER = 0.2 (600/(L-30) + 16)
NOTE: PROPOSED IMPACT EQUATION IS SHOWN FOR SPAN LENGTH OF 110 FT. WITH GIRDERS AT 10-FT. CENTERS
THROUGH AND DECK PLATE GIRDERS
90 TO 140-FT. SPANS
TOTAL IMPACTS
ROLLING EQUIPMENT WITHOUT HAMMER BLOW

Fig. 12. Recorded Impacts in Long Girder Spans Produced by Diesel Locomotives.

impacts in the ballasted floor bridges would be considerably smaller than those in open floor bridges; however, as shown in most of these diagrams, the impacts are about the same. A study of the following factors producing impact confirms the test results:

1. The rolling of the spring-borne weight of the locomotive about a longitudinal axis increases the load on one rail with a corresponding decrease on the other rail. There is no reason to believe that this rolling effect would be smaller for the ballasted floor bridges and the analysis of the data for rolling effects indicates it is about the same.

2. The increase in the axle loads due to the centrifugal force resulting from the loaded axles running over the deflected span should result in higher impact values in the ballasted floor bridges due to greater deflection.

3. The increase or decrease in axle loads resulting from the acceleration of the unsprung weight of the locomotives during the vertical oscillation should be the same for both open and ballasted floor bridges.

4. The resultant weights in the driving wheels of the steam locomotives, out-of-round wheels, flat spots, battered rail joints, and uneven track produce forced vibration of the structure and here the extra weight of the ballast reduces somewhat the synchronous speed with a corresponding reduction in the disturbing force and impact.

In addition to the total impacts, the analysis of the data secured on the girder spans indicate the following:

1. The stresses in the top flanges of deck spans are about the same as those in the lower flanges of symmetrical girders carrying open or ballasted timber floors. In girder spans with poured-in-place concrete floors, the stresses in the top flange are considerably lower than those in the bottom flange even though no effort was made to bond the concrete to the steel.

2. The stresses in the lower flanges of through spans with stringers and floor-beams are lower than those in the upper flanges, indicating that there is inter-action between the stringers, lateral bracing and lower flanges.

3. There is considerable variation between the stresses recorded on the inner and outer sides of the girder flanges indicating transverse bending.

4. The recorded total impacts in the web plates at the ends of the girders are somewhat higher than those recorded in the flanges at the center of the span.

5. The stresses in the lateral bracing are quite low but vary from a tensile stress to a compressive stress, which undoubtedly accounts for the large number of fatigue in these members.

Truss Spans

While an extensive program of testing truss spans has not yet been started, the research has secured readings on a few spans, such as the one shown on Fig. 14. The testing of a large truss span requires considerable study and planning for the location of the gages. It has been our experience that the usual truss member is subjected to considerable bending about both the horizontal and vertical axis so we usually place at least four gages on each member to arrive at the direct stress in the member. The gages are also usually

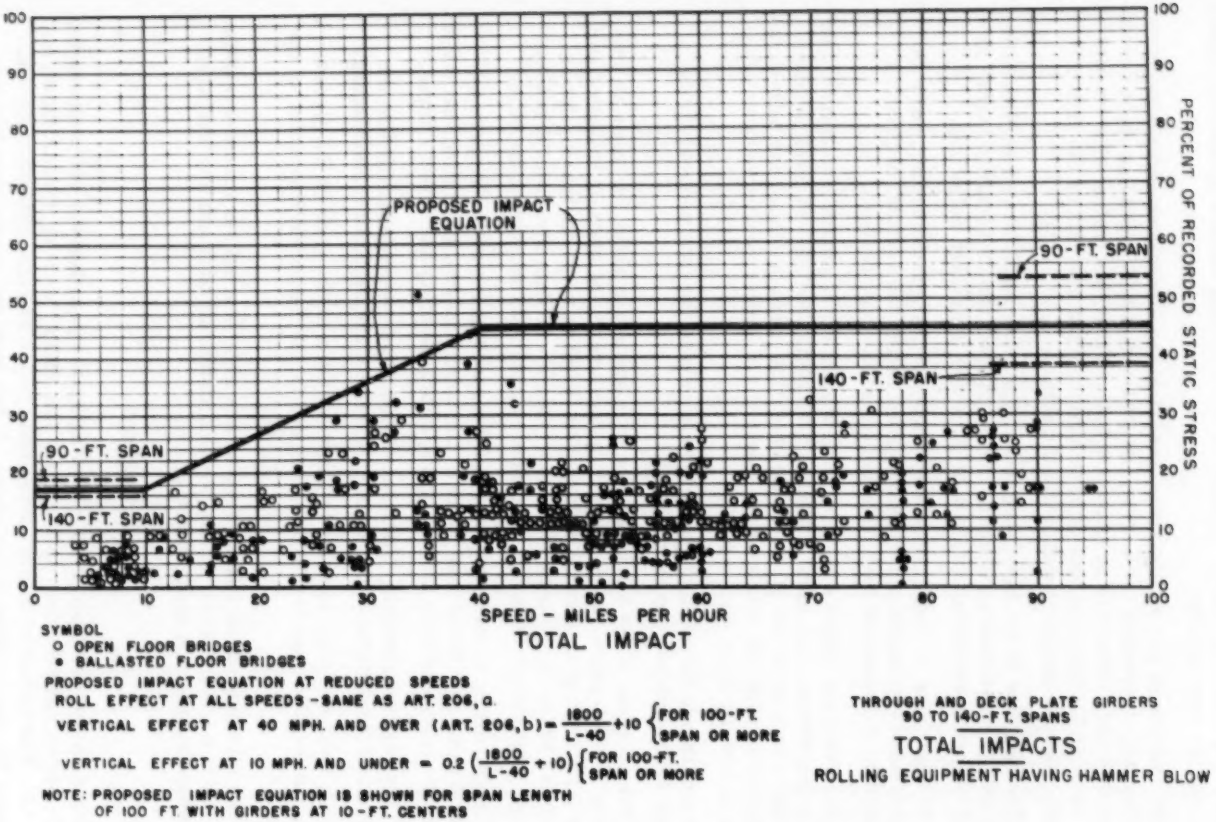

PROPOSED IMPACT EQUATION
90-FT. SPAN
140-FT. SPAN
90-FT. SPAN
140-FT. SPAN
PERCENT OF RECORDED STATIC STRESS
SPEED - MILES PER HOUR
TOTAL IMPACT
SYMBOL
○ OPEN FLOOR BRIDGES
● BALLASTED FLOOR BRIDGES
PROPOSED IMPACT EQUATION AT REDUCED SPEEDS
ROLL EFFECT AT ALL SPEEDS - SAME AS ART. 206, a.
VERTICAL EFFECT AT 40 MPH. AND OVER (ART. 206, b) = 1800/(L-40) + 10 {FOR 100-FT. SPAN OR MORE
VERTICAL EFFECT AT 10 MPH. AND UNDER = 0.2 (1800/(L-40) + 10) {FOR 100-FT. SPAN OR MORE
NOTE: PROPOSED IMPACT EQUATION IS SHOWN FOR SPAN LENGTH OF 100 FT. WITH GIRDERS AT 10-FT. CENTERS
THROUGH AND DECK PLATE GIRDERS 90 TO 140-FT. SPANS
TOTAL IMPACTS
ROLLING EQUIPMENT HAVING HAMMER BLOW

Fig. 13. Recorded Impacts in Long Girder Spans Produced by Steam Locomotives.

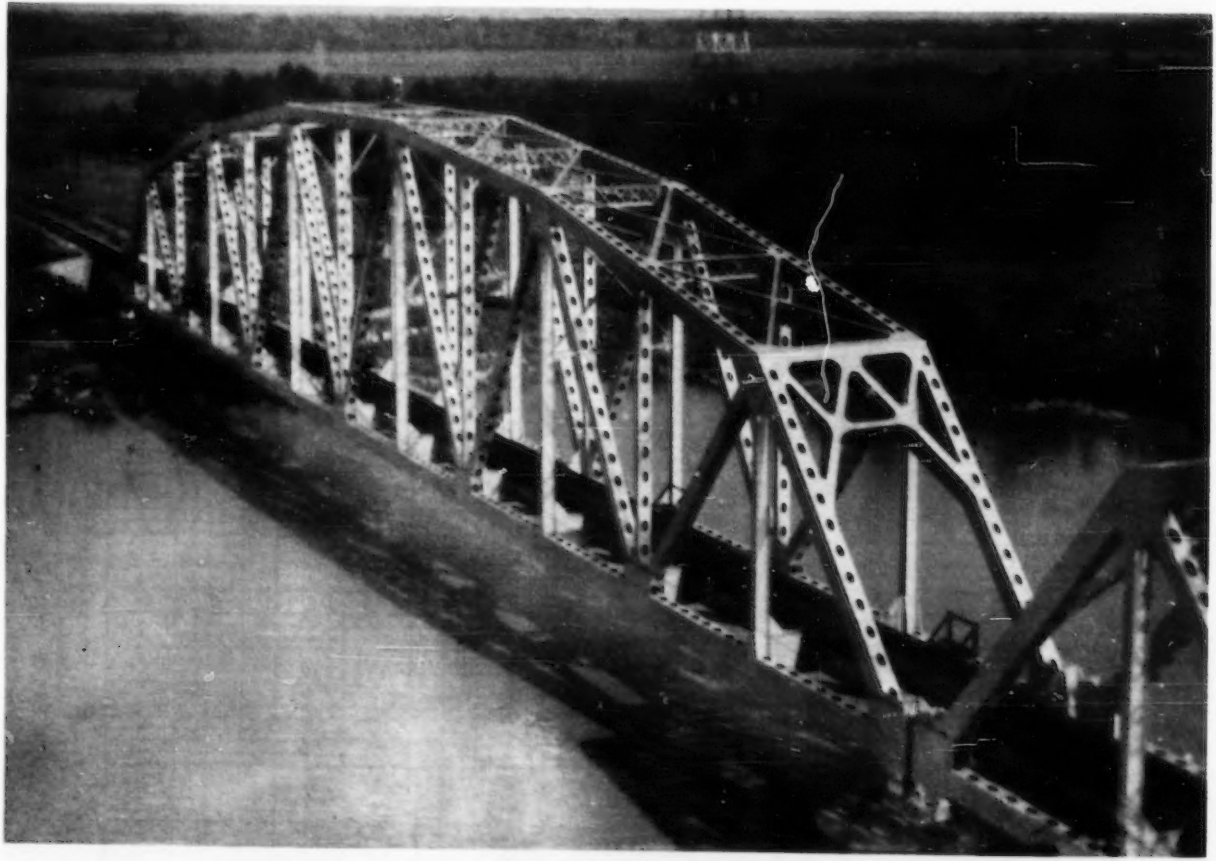


Fig. 14. General View of Long Truss Span.

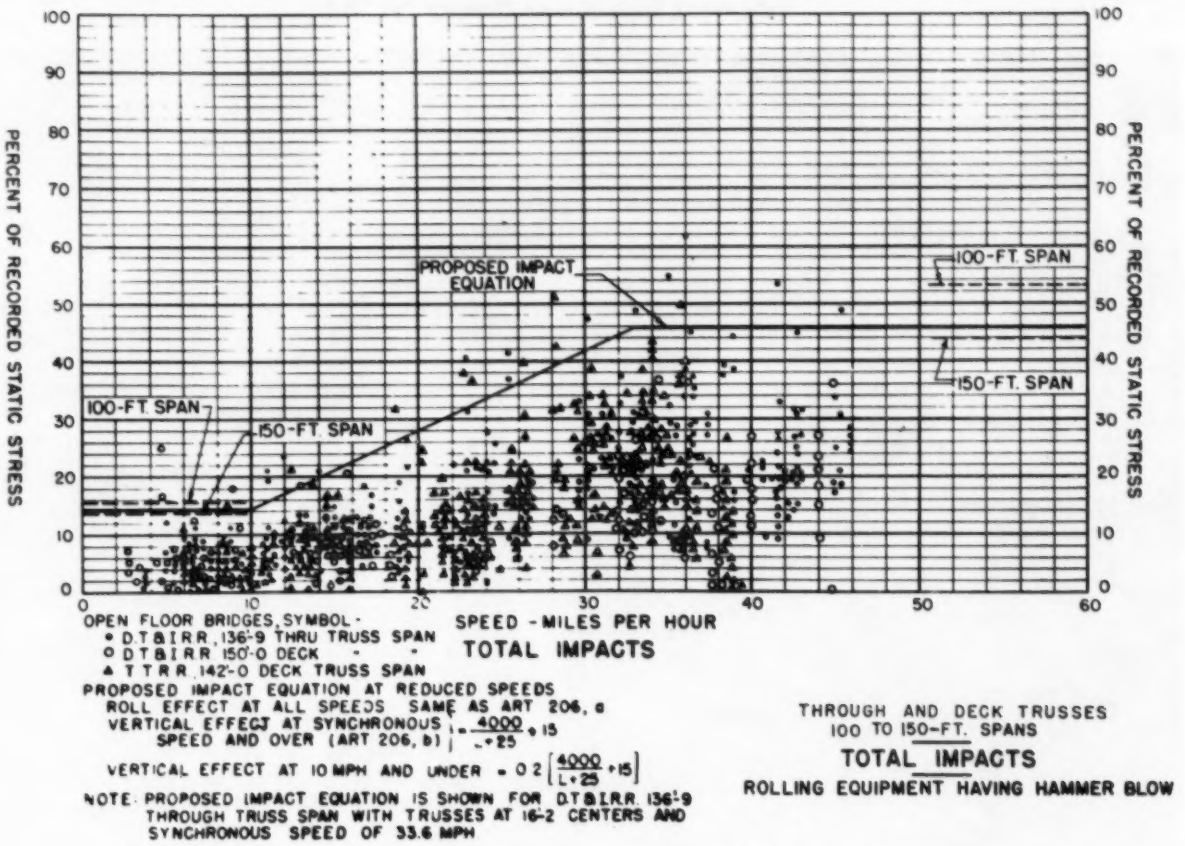

PERCENT OF RECORDED STATIC STRESS
PERCENT OF RECORDED STATIC STRESS
PROPOSED IMPACT EQUATION
100-FT. SPAN
150-FT. SPAN
100-FT. SPAN
150-FT. SPAN
SPEED - MILES PER HOUR
OPEN FLOOR BRIDGES, SYMBOL -
• D.T.&I.R.R. 136'-9 THRU TRUSS SPAN
○ D.T.&I.R.R. 150'-0 DECK " "
▲ T.T.R.R. 142'-0 DECK TRUSS SPAN
PROPOSED IMPACT EQUATION AT REDUCED SPEEDS
ROLL EFFECT AT ALL SPEEDS SAME AS ART 206, c
VERTICAL EFFECT AT SYNCHRONOUS SPEED AND OVER (ART 206, b) = 4000/(L+25) + 15
VERTICAL EFFECT AT 10 MPH AND UNDER = 0.2 [4000/(L+25) + 15]
NOTE: PROPOSED IMPACT EQUATION IS SHOWN FOR D.T.&I.R.R. 136'-9 THROUGH TRUSS SPAN WITH TRUSSES AT 16'-2 CENTERS AND SYNCHRONOUS SPEED OF 33.6 MPH
TOTAL IMPACTS
THROUGH AND DECK TRUSSES
100 TO 150-FT. SPANS
TOTAL IMPACTS
ROLLING EQUIPMENT HAVING HAMMER BLOW

Fig. 15. Recorded Impacts in Truss Spans Produced by Steam Locomotives.

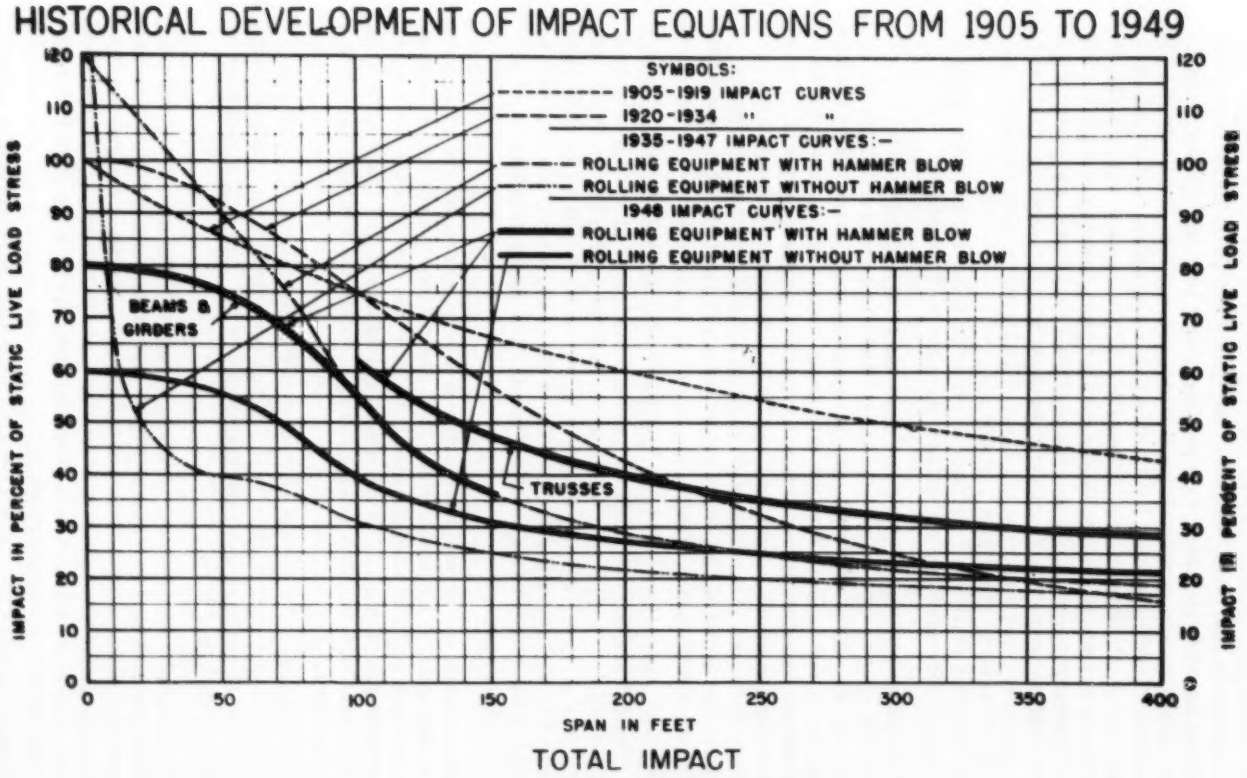

HISTORICAL DEVELOPMENT OF IMPACT EQUATIONS FROM 1905 TO 1949
SYMBOLS:
1905-1919 IMPACT CURVES
1920-1934 " "
1935-1947 IMPACT CURVES:-
ROLLING EQUIPMENT WITH HAMMER BLOW
ROLLING EQUIPMENT WITHOUT HAMMER BLOW
1948 IMPACT CURVES:-
ROLLING EQUIPMENT WITH HAMMER BLOW
ROLLING EQUIPMENT WITHOUT HAMMER BLOW
BEAMS & GIRDERS
TRUSSES
IMPACT IN PERCENT OF STATIC LIVE LOAD STRESS
IMPACT IN PERCENT OF STATIC LIVE LOAD STRESS
0 10 20 30 40 50 60 70 80 90 100 110 120
0 50 100 150 200 250 300 350 400
SPAN IN FEET
TOTAL IMPACT

Fig. 16. Historical Development of Impact Equations.

placed away from all rivets and other stress-raisers so that the stress on the gross section is measured.

The strain gage measurements secured on the truss spans tested have not been completely analyzed. However, the data secured on three truss spans varying in length from 136-ft 9-in to 150-ft have been completely analyzed and the total impacts recorded under steam locomotives are shown on Fig. 15. These three structures were built at the beginning of the century and were designed for light locomotives. Since the bridges are now carrying locomotives whose weights are about 50 per cent greater than the locomotive for which the bridges were designed, the particular railroads requested the research staff to conduct tests on these bridges at their expense, to determine if the stresses resulting from the heavier locomotives exceeded the increased stresses allowed by the AREA rating rules.

The detailed analysis of the data secured on these three truss spans and from others only partially completed, indicate the following:

1. The maximum total impacts, as shown in this diagram, occur at a relatively low speed. The impact values plotted were recorded in the various chord and web members of the three truss spans as indicated by the three symbols. It can be seen that the maximum impact in any one span attained a maximum and then decreased with a further increase in speed. For example, the maximum impact in the 136-ft 9-in span as indicated by the solid circles, occurred at a speed of 36 mph or 3.2 rps and then decreased. This is due to the speed of the locomotive drivers synchronizing with the natural period of vibration of a span.

2. The recorded static live load stresses in the top chords of through spans are greater than the calculated stresses while those in the lower chords are smaller than the calculated. This appears to be due to the stringers taking part of the stresses usually assumed to be carried entirely by the lower chord. The stringers are usually above the lower chord so the result is a change in the effective depth of the truss. The opposite effect has been found in deck spans.

3. The impacts in the various members of a truss are not the same and are not a maximum at the same time. The reason for this variation in impact has not been definitely determined but since this phenomenon does exist, it eliminates the possibility of determining impacts in any truss span by deflection readings, as such readings only indicate the average impact in all the members.

4. The stresses in the individual eye-bars of any one member of a pin-connected truss span usually vary. It has been found that in some members one or two of the bars were not taking any of the load with a resulting overstress in the other bars.

5. The secondary or bending stresses in some of the members of the older bridges are sometimes so large that compression has been found in one corner of a lower chord tension member with tension in one corner of the end post. Even in pin-connected trusses, where the pins were undoubtedly frozen, a zero stress has been found on one edge of a bar with twice the average stress on the other edge.

The elimination or reduction of these high bending stresses in tension members is very important on account of fatigue failures. Several railroads have already experienced failures in tension members of their older bridges which were apparently the result from high bending plus direct stresses.

6. The eccentricity in some of the compression members has been found greater than we normally expect and this eccentricity has been found to exist about both axes of the member. The eccentricity in any compression member materially reduces its carrying capacity.

7. As previously mentioned, the elasticity of the supports for any structure plays a very important part in the amount of impact in the structure. The low recorded impact values in the 150-ft truss span, shown by the open circles in the diagram, indicate that the high steel viaduct columns supporting this span were acting as springs and produced more than the usual amount of damping. In another long truss span, the recorded impacts increased after the slender concrete piers supporting the trusses had been reinforced.

Impacts in stringers supported by elastic floorbeams and floorbeams supported by elastic hangers, are always lower than those in the same length spans supported by concrete piers or abutments. However, floorbeam hangers, which are supported by more rigid main chord members of the truss, receive about the same impact as a span with rigid supports.

Present Impact Equations

The railroad bridge engineers realized in 1948 that, while the study of impact in bridges was not complete, there appeared to be sufficient evidence, to warrant a change in the design and rating rules values. It was evident that previous design values for the shorter spans were entirely too high while those for the design of trusses were too low. Accordingly, AREA Committee 15, Iron and Steel Structures recommended the proposed values shown on the previous slides and the adopted impact values for the design of steel structures is shown by the heavy lines on Fig. 16. The lower curves show the values produced by the diesel locomotives while the two upper curves are the values produced by the steam locomotives. Future changes in these impact values can be expected as more data is accumulated, especially on girder and truss spans.

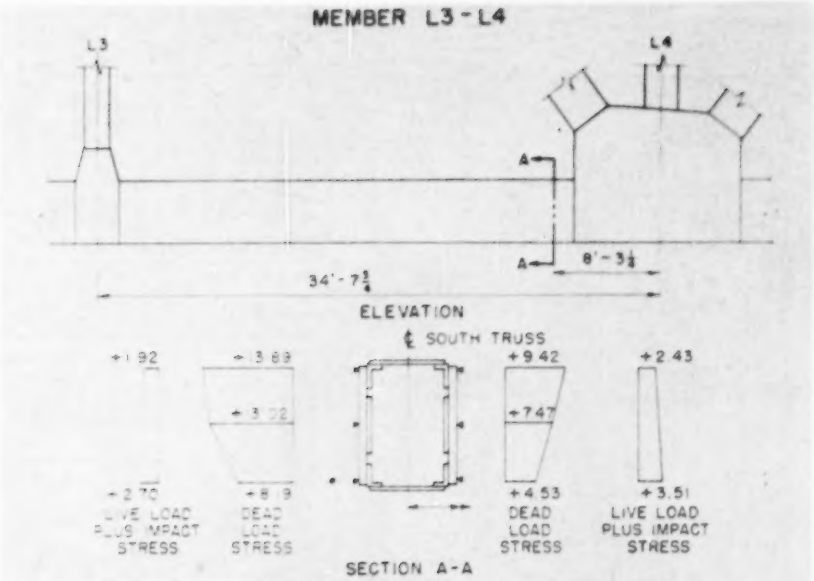
In addition to the 1948 impact curves, this figure also shows the various impact values that have been in use by the railroad bridge engineers since the first equation was adopted in 1905.

Secondary Stresses

The construction of the large double track railway bridge shown on Fig. 17, with 350-ft deck truss spans having a ballasted timber floor by one of the western railroads afforded an opportunity for a complete study of the dead and live load stresses with particular attention to the secondary effects resulting from the rotation of the joints in the main members of these spans. Accordingly, the research staff took over 5,000 Berry strain gage readings on about 700 gage lines at various locations on the truss members while the spans were being assembled for reaming in the fabricating shop and a like number on the gage lines after the bridge had been completed. Extensive electrical strain gage readings were also taken under both diesel and steam locomotives operating at various speeds over the bridge so that the live load direct, secondary and impact effects could be determined.

The data secured during these test on lower chord member L_3-L_4, see Fig. 18, indicates that a maximum dead load stress of 13.89 ksi tension was recorded in one corner of lower chord L_3-L_4, as compared with an average

Fig. 17. General View of 350 ft Double Track Truss Span With Ballasted Track.



	DEAD LOAD STRESSES		LIVE LOAD STRESSES	
	NORTH TRUSS	SOUTH TRUSS	NORTH TRUSS	SOUTH TRUSS
CALCULATED AVERAGE STRESS	+6.51	+6.51		+3.16
RECORDED AVERAGE STRESS	+7.76	+9.01		+2.64
RECORDED MAXIMUM STRESS	+10.89	+13.89		+3.51
BENDING STRESS - HOR. AXIS	2.79	2.65		0.47
" " - VER. AXIS	0.25	2.03		0.33
" " - BOTH AXES	3.13	4.88		0.87
PERCENT BENDING - HOR. AXIS	35.9	29.4		17.8
" " - VER. AXIS	3.2	22.5		12.5
" " - BOTH AXES	40.3	54.1		32.9

Fig. 18. Secondary and Direct Stresses in Lower Chord Member.

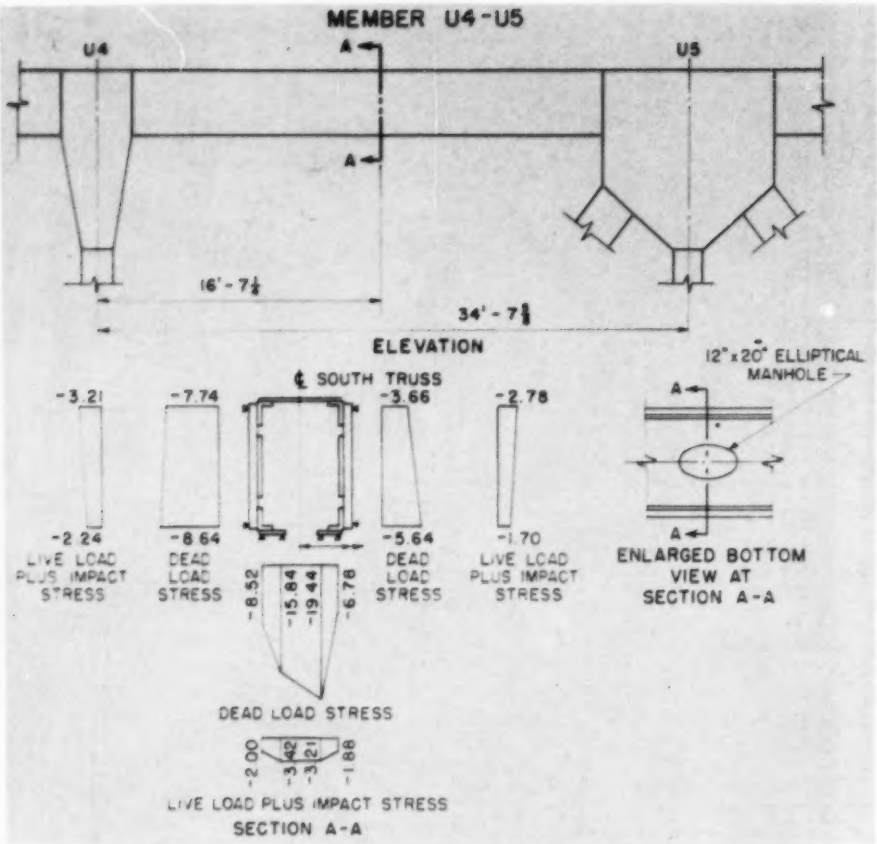


Fig. 19. Secondary and Direct Stresses in Top Chord Member.

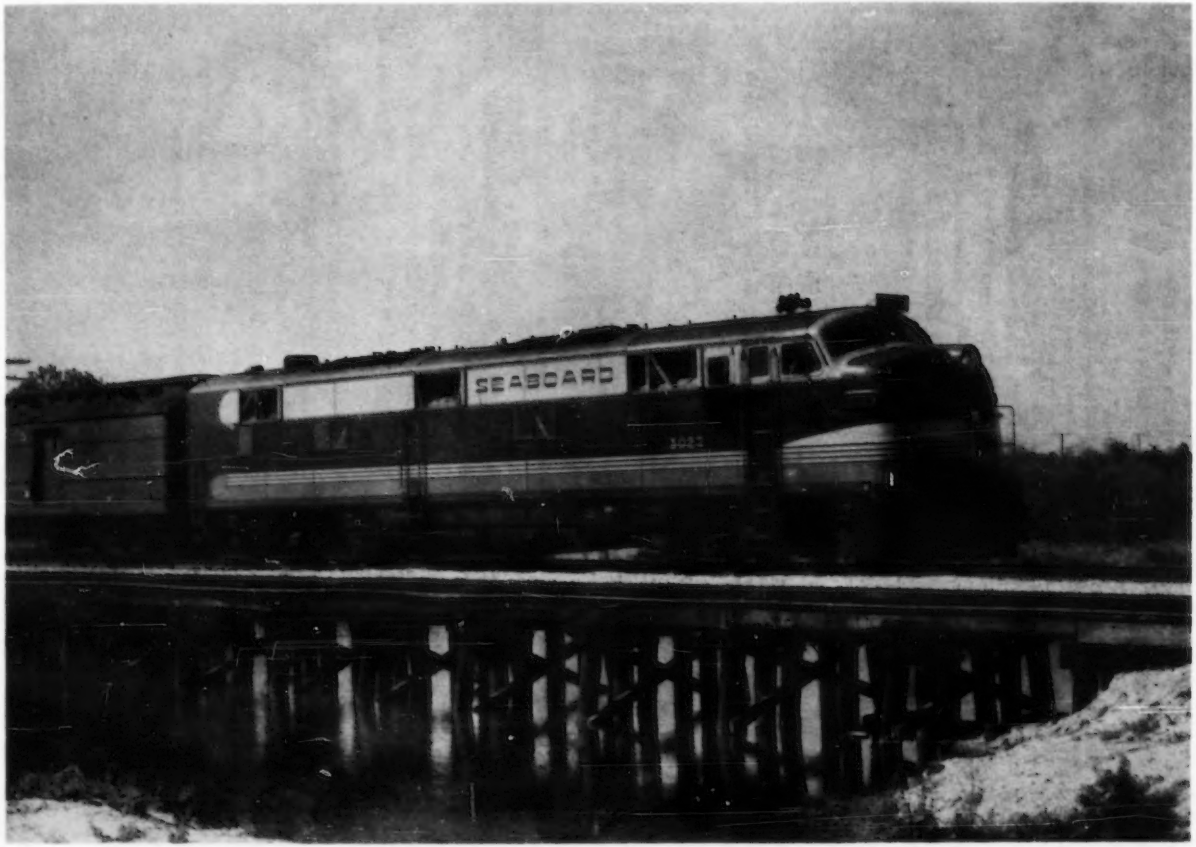


Fig. 20. General View of Timber Trestle.

calculated stress of 6.51 ksi. The data indicates that bending was taking place about both axes of the various truss members and for member L_3-L_4, the bending about the horizontal axis was 29.4 per cent for the dead load and 17.8 per cent for the live load. The bending about the vertical axis was 22.5 per cent for the dead load and 12.5 per cent for the live load. The total bending about both axes was 54.1 per cent for the dead load and 32.9 per cent for the live load.

The chord members of this bridge consisted of box members with elliptical man-holes. Dead load and live load readings were taken on a section through the man-holes and the stresses found at the various gage locations in top chord member U_4U_5 are shown on Fig. 19. A dead load stress of 19.44 ksi was measured at the edge of the man-hole which is about three times the average stress on the gross section of the member. Laboratory investigations of the stresses around man-holes in similar members have indicated stress raising factors about the same as found during these tests.

Timber Bridges

There is a total length of about 1800 miles of timber railroad bridges in the United States today which is about the same as the total length of steel railroad bridges. We have conducted tests on a few of these bridges such as that shown in Fig. 20, and the data collected has been the basis for a large program of laboratory and field tests which we are just starting.

The present design of timber railroad structures is based on static loads only, as previous laboratory investigations have shown that timber has twice the strength under suddenly applied loads as it has under the same load applied statically. However, it is now known that the stress under a suddenly applied load goes from zero to a maximum in about 1/100 of a second while under a high speed locomotive it takes about 3/10 of a second for the stress to reach a maximum, or about 30 times longer.

The field testing consists essentially of determining the magnitude of the stresses in the stringers and piles, the time of duration of these stresses and the frequency of occurrence of the maximum stresses so that these values can be correlated with laboratory results. It is becoming apparent that the dynamic or impact effects should be considered if we are to place the design of timber structures on a scientific basis.

The laboratory investigation of timber stringers under repeated loadings has been started and one phase of this program is the testing of full size 8 in by 16 in by 14 ft long bridge stringers as taken from the storage yards. Another phase is the laboratory testing of 1/4 scale clear stringers under rigid control of all the variables.

The data secured from these field and laboratory tests will enable us to eventually design timber structures on a more scientific and economical basis.

Concrete Bridges

A report has recently been published by the AREA covering an investigation of the static and dynamic stresses in four reinforced concrete railroad bridges, consisting of three pile trestles with pre-cast slabs, as shown in Fig. 21, and one continuous 4-span structure. The strains in the concrete and reinforcing bars were determined under a total of 686 diesel and steam locomotive test runs over a complete range of speeds. The results of these tests indicate the following:

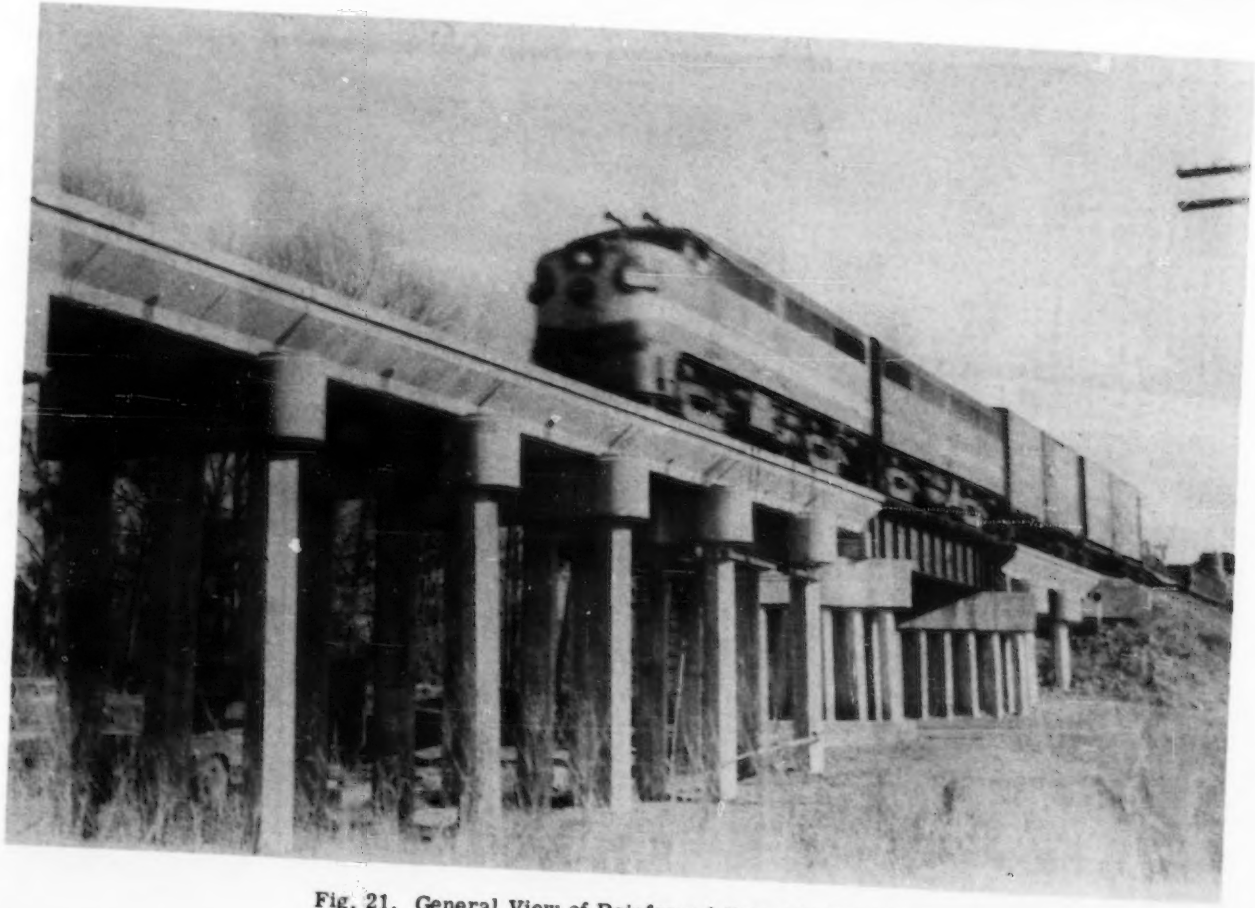


Fig. 21. General View of Reinforced Concrete Trestle.

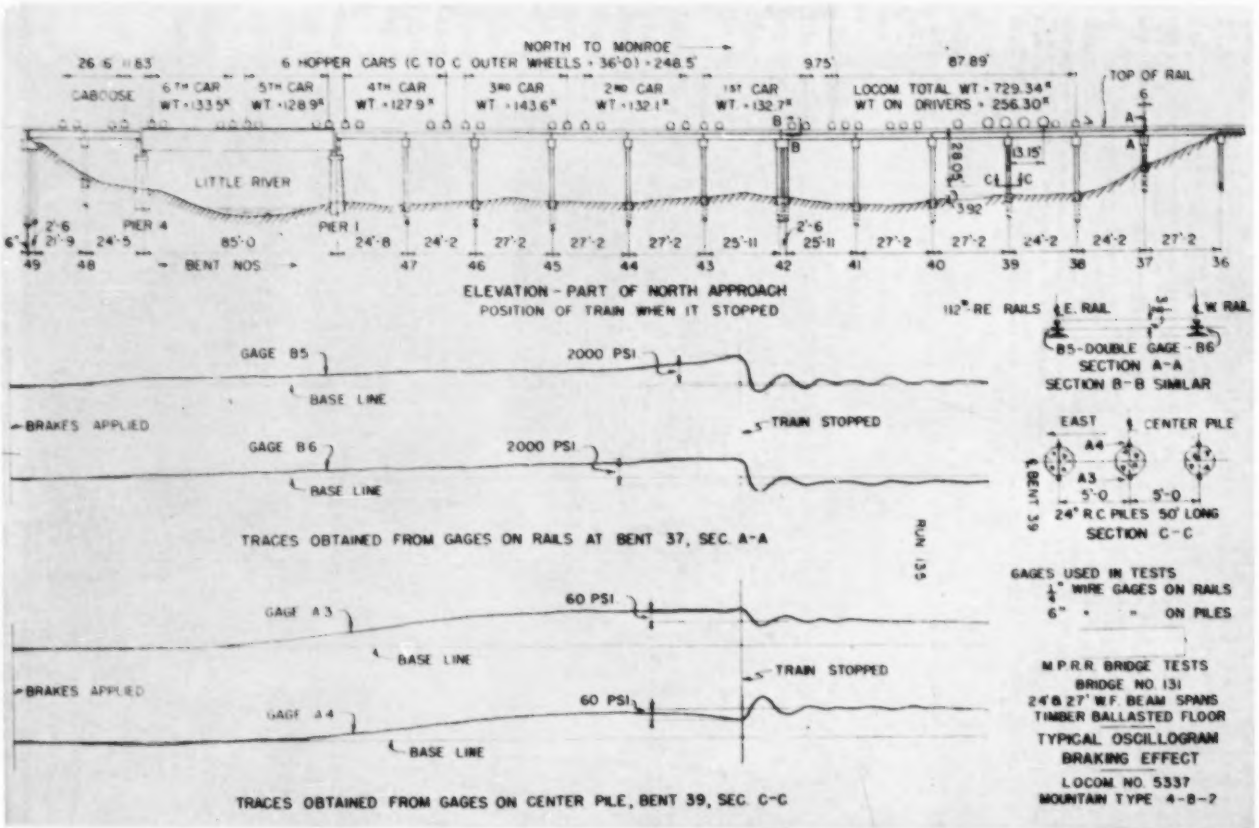


Fig. 22. Stresses Produced in Concrete Piles by Braking and Traction of Locomotives.

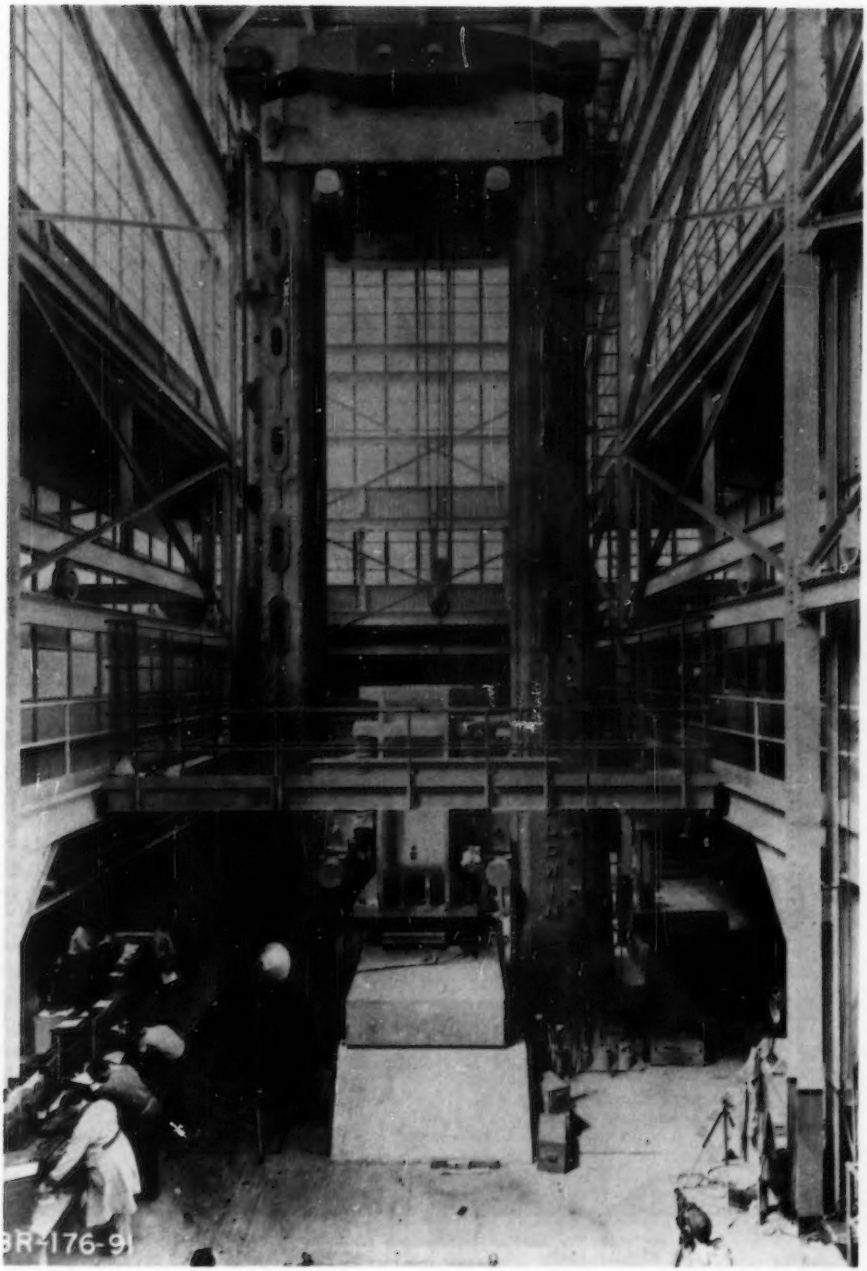


Fig. 23. General View of 5,000,000 lb. Testing Machine Used to Test Full Size Concrete Slabs.

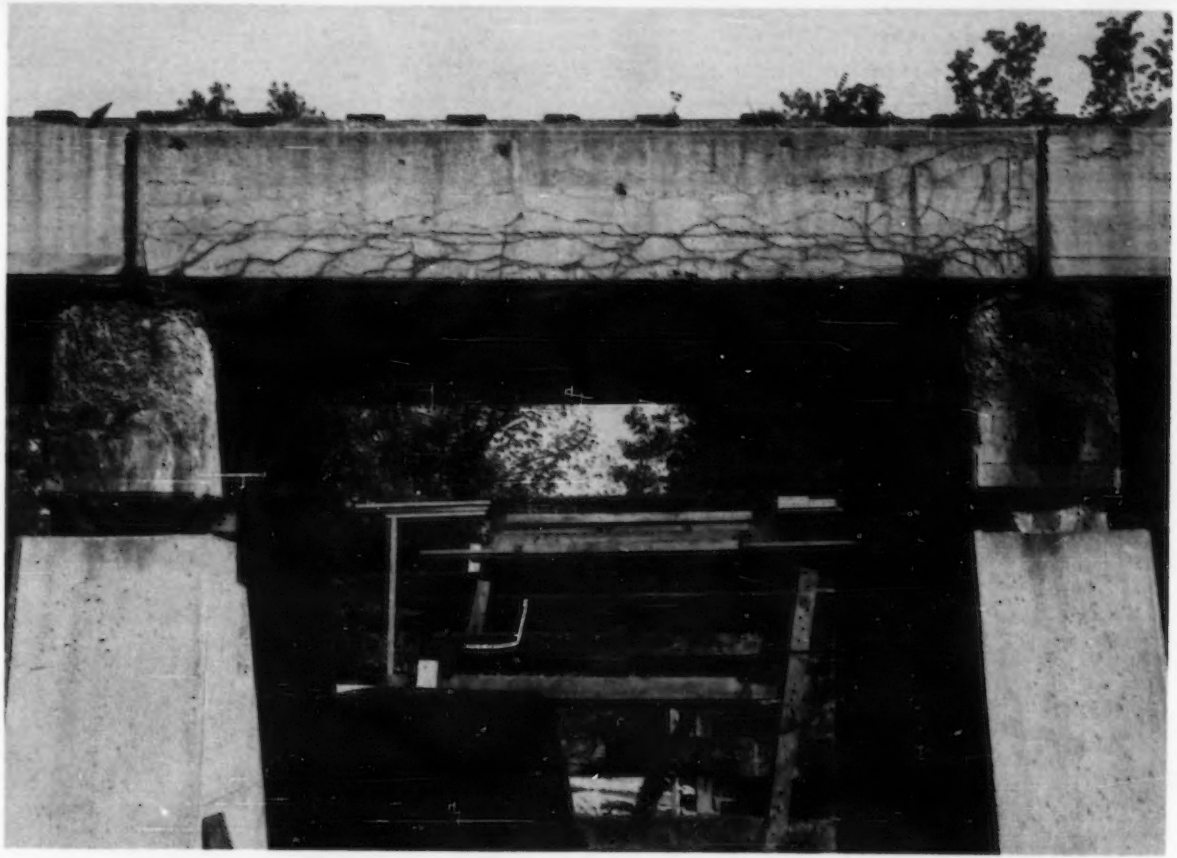


Fig. 24. General View of Deteriorated Reinforced Concrete Slabs Before Removal for Testing to Failure.

1. The simultaneous strains in the concrete and reinforcement on the tension side of the slabs showed that the concrete was carrying part of the tensile stresses.

2. The recorded static stresses in the compressive concrete were appreciably lower than the calculated stresses, based on the usual assumption that the concrete is not taking any tension, and were generally lower than those calculated on the basis that the concrete is taking tension.

3. The impact percentages in the compression concrete were greater than the AREA allowance, but the recorded maximum stresses were only about 80 per cent of the calculated live loads plus impact stresses.

4. While the impact percentages in the tensile reinforcement were greater than the AREA allowance, the recorded maximum stresses were only about 20 per cent of the calculated live load plus impact stresses.

The testing of these trestles afforded an opportunity to secure data on the bending stresses in the piles resulting from a train crossing the bridge at various speeds under normal operating conditions as compared to the bending stresses produced by the same train crossing the bridge with the brakes applied. In addition to the gages on the piles, gages were also placed longitudinally on the web of the rails to determine the magnitude of any longitudinal forces carried by the rails.

The general character of the stresses produced in the piles and rails by stopping the train on the bridge by a hard service application of the brakes is shown by the traces on Fig. 22. The general elevation of the bridge is shown at the top with the locomotive and car wheels shown in the position where the train stopped. The upper two traces, gages B5 and B6, were located on the rails over bent 37 ahead of the locomotive. It can be seen that the rails started carrying a longitudinal force when the brakes were first applied, and this force kept increasing until the forward movement of the train stopped. As soon as the train stopped, the longitudinal forces in the rails were released, but it can be seen that the structure continued to vibrate at a low frequency for some time. A maximum compression stress of 2000 psi was recorded in each rail and assuming uniform distribution of stress over the entire rail cross section, a total longitudinal force of 44,000 lb. was carried by the two rails for this particular run.

The two lower traces, gages A3 and A4, were located on opposite sides of the center of bent 39 at Section C-C. It can be seen that the action of the piles was similar to that of the rails, except that the piles carried vertical load as well as longitudinal forces. For the particular run shown, the north side of the piles, gage A3, was stressed in bending to 60 psi compression and the south side was stressed to 60 psi tension by the braking force. By using the average bending stress in the three piles and considering the bent fixed at the top and bottom, it was determined that bent 39 was carrying a longitudinal force of 5100 lb.

The low stresses recorded in reinforced concrete bridges under moving trains raised the question as to their actual ultimate strength and corresponding factor of safety. Arrangements were made with the Bureau of Reclamation in Denver for the use of their 5,000,000 lb. testing machine shown in Fig. 23, to determine the actual carrying capacity of full size concrete slabs designed to carry railroad loading. A total of six slabs were tested in this machine and strains in the concrete and reinforcement over a complete range of load were recorded. The slabs tested consisted of:

Two old slabs removed from a railroad bridge after being in service for over 40 years.

One new slab designed in accordance with current AREA Specifications.

Two new slabs designed in accordance with the Ultimate-load theory.

One new slab, prestressed by pretensioning in accordance with current recommended practice.

While all the new slabs carried more load than was expected, the load carrying capacity of the two old slabs far exceeded the expected load. The two old slabs shown in Fig. 24 while still in the railroad bridge had many of the reinforcing bars exposed and the concrete had spelled off at many locations in the slabs. The concrete was badly deteriorated along the inside edge and this deterioration extended in about 1 ft from the edge. In spite of all the spalling and deterioration, the slabs carried more than 3 1/2 times their original design load including impact about 2 1/2 times the present design loads including impact.

CONCLUSION

In conclusion, the vast amount of research being conducted in this country on structural problems should eventually enable the structural engineer to determine more precisely the actual behavior of structures subjected to moving loads and to design these structures more economically.

PROCEEDINGS PAPERS

The technical papers published in the past year are presented below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways (WW) divisions. For titles and order coupons, refer to the appropriate issue of "Civil Engineering" or write for a cumulative price list.

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JULY: 457(AT), 458(AT), 459(AT)[c], 460(IR), 461(IR), 462(IR), 463(IR)[c], 464(PO), 465(PO)[c].

AUGUST: 466(HY), 467(HY), 468(ST), 469(ST), 470(ST), 471(SA), 472(SA), 473(SA), 474(SA), 475(SM), 476(SM), 477(SM), 478(SM)[c], 479(HY)[c], 480(ST)[c], 481(SA)[c], 482(HY), 483(HY).

SEPTEMBER: 484(ST), 485(ST), 486(ST), 487(CP)[c], 488(ST)[c], 489(HY), 490(HY), 491(HY)[c], 492(SA), 493(SA), 494(SA), 495(SA), 496(SA), 497(SA), 498(SA), 499(HW), 500(HW), 501(HW)[c], 502(WW), 503(WW), 504(WW)[c], 505(CO), 506(CO)[c], 507(CP), 508(CP), 509(CP), 510(CP), 511(CP).

OCTOBER: 512(SM), 513(SM), 514(SM), 515(SM), 516(SM), 517(PO), 518(SM)[c], 519(IR), 520(IR), 521(IR), 522(IR)[c], 523(AT)[c], 524(SU), 525(SU)[c], 526(EM), 527(EM), 528(EM), 529(EM), 530(EM)[c], 531(EM), 532(EM)[c], 533(PO).

NOVEMBER: 534(HY), 535(HY), 536(HY), 537(HY), 538(HY)[c], 539(ST), 540(ST), 541(ST), 542(ST), 543(ST), 544(ST), 545(SA), 546(SA), 547(SA), 548(SM), 549(SM), 550(SM), 551(SM), 552(SA), 553(SM)[c], 554(SA), 555(SA), 556(SA), 557(SA).

DECEMBER: 558(ST), 559(ST), 560(ST), 561(ST), 562(ST), 563(ST)[c], 564(HY), 565(HY), 566(HY), 567(HY), 568(HY)[c], 569(SM), 570(SM), 571(SM), 572(SM)[c], 573(SM)[c], 574(SU), 575(SU), 576(SU), 577(SU), 578(HY), 579(ST), 580(SU), 581(SU), 582(Index).

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JANUARY: 583(ST), 584(ST), 585(ST), 586(ST), 587(ST), 588(ST), 589(ST)[c], 590(SA), 591(SA), 592(SA), 593(SA), 594(SA), 595(SA)[c], 596(HW), 597(HW), 598(HW)[c], 599(CP), 600(CP), 601(CP), 602(CP), 603(CP), 604(EM), 605(EM), 606(EM)[c], 607(EM).

FEBRUARY: 608(WW), 609(WW), 610(WW), 611(WW), 612(WW), 613(WW), 614(WW), 615(WW), 616(WW), 617(IR), 618(IR), 619(IR), 620(IR), 621(IR)[c], 622(IR), 623(IR), 624(HY)[c], 625(HY), 626(HY), 627(HY), 628(HY), 629(HY), 630(HY), 631(HY), 632(CO), 633(CO).

MARCH: 634(PO), 635(PO), 636(PO), 637(PO), 638(PO), 639(PO), 640(PO), 641(PO)[c], 642(SA), 643(SA), 644(SA), 645(SA), 646(SA), 647(SA)[c], 648(ST), 649(ST), 650(ST), 651(ST), 652(ST), 653(ST), 654(ST)[c], 655(SA), 656(SM)[c], 657(SM)[c], 658(SM)[c].

APRIL: 659(ST), 660(ST), 661(ST)[c], 662(ST), 663(ST), 664(ST)[c], 665(HY)[c], 666(HY), 667(HY), 668(HY), 669(HY), 670(EM), 671(EM), 672(EM), 673(EM), 674(EM), 675(EM), 676(EM), 677(EM), 678(HY).

MAY: 679(ST), 680(ST), 681(ST), 682(ST)[c], 683(ST), 684(ST), 685(SA), 686(SA), 687(SA), 688(SA), 689(SA)[c], 690(EM), 691(EM), 692(EM), 693(EM), 694(EM), 695(EM), 696(PO), 697(PO), 698(SA), 699(PO)[c], 700(PO), 701(ST)[c].

JUNE: 702(HW), 703(HW), 704(HW)[c], 705(IR), 706(IR), 707(IR), 708(IR), 709(HY)[c], 710(CP), 711(CP), 712(CP), 713(CP)[c], 714(HY), 715(HY), 716(HY), 717(HY), 718(SM)[c], 719(HY)[c], 720(AT), 721(AT), 722(SU), 723(WW), 724(WW), 725(WW), 726(WW)[c], 727(WW), 728(IR), 729(IR), 730(SU)[c], 731(SU).

JULY: 732(ST), 733(ST), 734(ST), 735(ST), 736(ST), 737(PO), 738(PO), 739(PO), 740(PO), 741(PO), 742(PO), 743(HY), 744(HY), 745(HY), 746(HY), 747(HY), 748(HY)[c], 749(SA), 750(SA), 751(SA), 752(SA)[c], 753(SM), 754(SM), 755(SM), 756(SM), 757(SM), 758(CO)[c], 759(SM)[c], 760(WW)[c].

c. Discussion of several papers, grouped by Divisions.

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